Contents

Executive Summary

The purpose of Technical Report 1 is to evaluate the existing conditions of a Hotel located in the Northeast United States. To perform this evaluation, figures and charts are used to describe the foundations, framing system, floor system, lateral system, and roof system. The codes used in design and analysis are compared and the materials are listed.

Using ASCE 7-05 and International Building Code 2009, gravity loads were investigated and matched the design loads used by the engineer of record. Also, three checks were performed to examine the sizes used. The first was the precast concrete plank used for the floor system in a typical guest room. By analyzing the amount of prestress in a plank with 6 strands at 6/16" diameter, it was determined that the plank was overprestressed and suitable to carry the loads. A W30x191 wide flange beam on the second floor was checked because it held the façade and a four story bearing wall as well. The beam was determined to be sized correctly and was controlled by the applied moment rather than deflection because most of the load was the masonry wall weight. Lastly, a W12x96 exterior column supporting the beam on the first floor was deemed adequate for the applied loads.

The snow loads for flat roof and drift against the parapet were matches to the design loads as well. There were no secondary drains on the roof, only a main drain along the center and scuppers at the base of the parapets. Because of this, a rain load analysis was performed per IBC 2009 and the roof plank was adequate to withstand a drain backup.

Lateral loads were determined using the Analytical Method for wind and by the Equivalent Lateral Force Method for seismic in ASCE 7-05. The parapet surrounded the entire roof added a significant pressure around the top of the building. Since the Hotel is a slender building, one direction was approximately four times greater than the other. A comparison of the base shear and overturning moment for both showed that seismic was almost double that of the wind. This is likely due to the fact that it is a very heavy building assembled with masonry and plank construction. The design base shear for seismic was about 50 kips less then what was evaluated in this report. However that was obtained using a computer model which will be more accurate than a hand analysis.

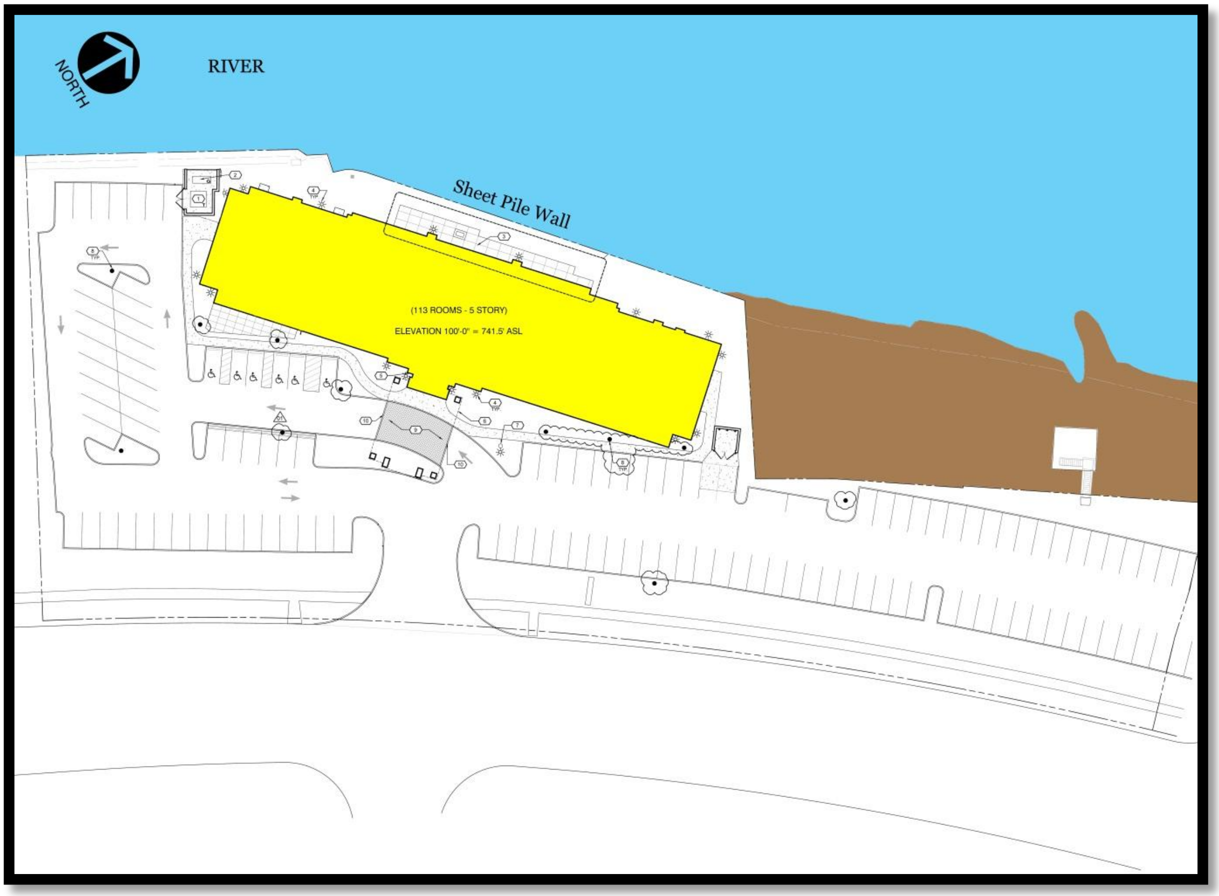
Introduction

Located along a river in the Northeast United States (henceforth referred to as Hotel N.E.U.S.), this five story, 113 room hotel is constructed with masonry bearing walls and a precast concrete floor system. It stands in place of an old steel mill and was constructed as part of the area's development in the 1990's.

At its tallest, the building is 60'-8" tall with a long slender shape that allows for windows in every room. Its façade consists of arching exterior insulation finishing system (EIFS) and a brick veneer. The warm colors of beige and brown provide a sense of comfort and soothing that communicate the architecture's purpose, a place to rest.

All of the amenities of a hotel are included, such as a pool, fitness area, meeting room, ADA accessible rooms, and sunlight for all rooms. There is an overhang at the entrance allowing for drop off and pick up with protection from the elements. The Hotel N.E.U.S. provides 75,209 ft^2 of floor area to a location lacking such facilities. Construction started in October of 2011 and is slated to finish in November of 2012 and cost $9.2 million dollars.

Note: The overhang at the entrance is not considered in the analysis or evaluation of this building at any point. Also, all photos/plans/documents provided by Atlantic Engineering Services



Structural Overview

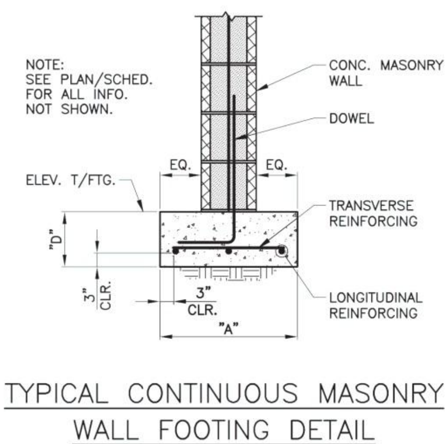
Foundations

Michael Baker Jr., Inc. provided the Geotechnical report in July of 2011. They included a history of the site that impacts the features below grade for this project. Pre-1986 the site of the Hotel N.E.U.S. was occupied by a steel mill. Cooling towers were located at the footprint of the current building while a gantry crane and tracks were to the Southwest. The sheet pile retaining wall was constructed in 1979. In 1990's a development of the area began and the mill was removed. Foundations and other below grade structures were usually removed to about to about one foot below grade. In 2001 a Damon's Restaurant and parking lot were constructed in the area that the hotel is to be located. Fill was added to the site during this time.

Geotechnical Consultants, Inc. drilled seven boring in April of 2001 to support Damon's Restaurant and those reports were included and mostly consisted of Slag and Concrete with little Silt. Terra Testing excavated four test pits and drilled thirteen test borings in April of 2011. They totaled 10 linear feet of rock and 282 linear feet of soil (see Figure 3 for location of all borings). The major finding in these tests was that there were buried concrete obstructions. They were determined to be the concrete pad that supported the cooling towers in the past.

The fill was considered to be suitable for a shallow spread foundation system. The bearing pressure was controlled by a limiting settlement of one inch and the capacity of the soil. The bearing capacity of the soil increases with the size of the footing. Larger footings cause much higher stresses however, so the bearing pressure decreases with larger sizes (see Figure 1 for tables providing various sizes). A minimum of a 3' x 3' reinforced footing was suggested and no less than 16.7' center-to-center distance between wall footings. Footings bearing on the concrete pad were allowed a reduction of 1.5'.

Continuous wall footings range from 2'-0" wide to 9'-0" wide with typically #5 or #7 for longitudinal and transverse reinforcement. Column footings ranged from 6'x6'x1'-6" to 8'x8'x1'-8" (see Figure 1 for footing schedule). Typical piers are 24"x24" with 4-#6 vertical with #3 at 12" ties.



CONTINUOUS WALL FOOTING SCHEDULE

MARK	WIDTH "A"	DEPTH "D"	LONGITUDINAL REINFORCING	TRANSVERSE REINFORCING	MARK
WF1	2'–0"	1'–0"	2–#5 CONT.	#5x1'–6" @ 24" O.C.	WF1
WF2	3'–0"	1'–0"	3–#5 CONT.	#5x2'–6" @ 24" O.C.	WF2
WF3	9'–0"	1'–6"	6–#7 CONT.	#7x8'–6" @ 12" O.C.	WF3
WF4	5'–0"	1'–0"	3–#7 CONT.	#7x4'–6" @ 18" O.C.	WF4
WF5	6'–0"	1'–3"	6–#5 CONT.	#5x5'–6" @ 18" O.C.	WF5

Figure 1: Continuous Masonry Wall Footing detail and schedule

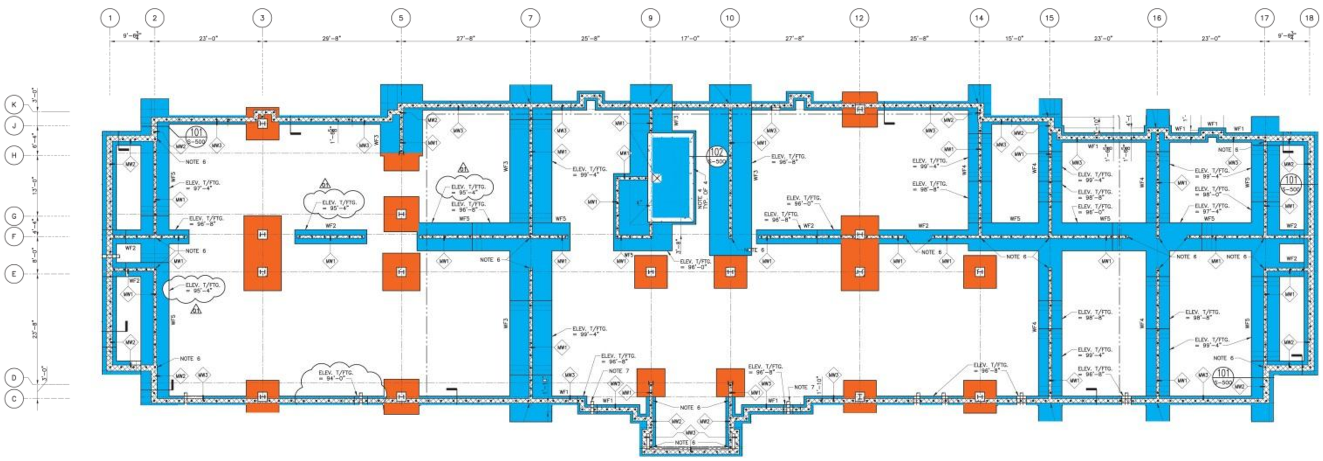


Figure 2: Foundation Plan.
Blue- wall footings
Orange- Column Footings

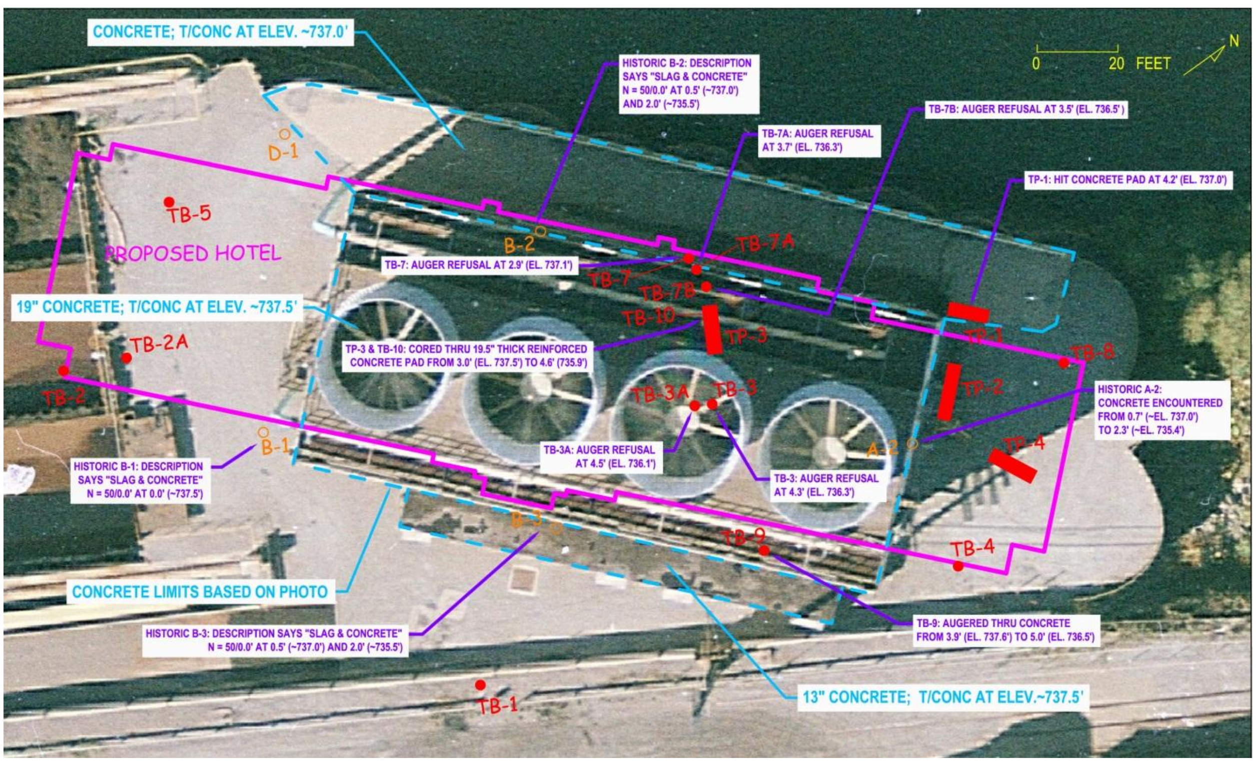


Figure 3: Site map showing test borings, existing mat foundation, hotel footprint, and location of former chilling towers.

Floor System

The floor system is composed of 8" Hollowcore precast concrete plank. There is a 3/4" topping to level off the floor since the planks have camber when they come out of production. The plank allows for long spans between the bearing walls. The smallest span is 15'-0" while the largest is 29'-8". Due to the large open spaces on the first floor, large transfer beams are used to carry the walls on the second floor up to the roof. These wide flange beams are approximately 30" in depth and weigh anywhere from 90 to 191 pounds per foot. Smaller beams span the corridor between walls and are much smaller, ranging from W6x25 to W24x68.

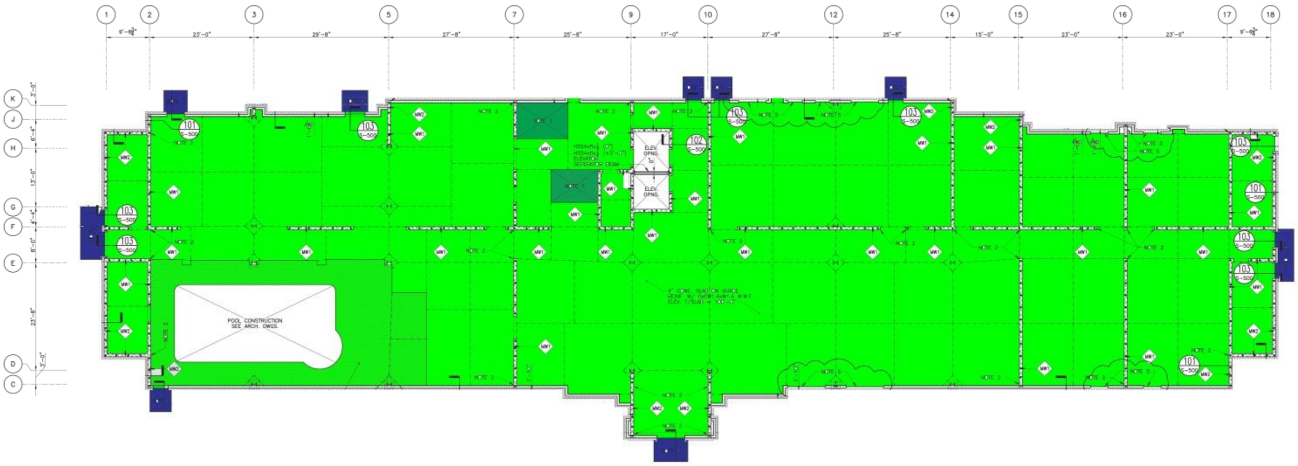


Figure 4: Slab on grade. Light green- 4" Conc. Slab on grade w/ 6x6W1.4xW1.4 W.W.F.

Dark Green- 3'-0" thick Conc. Slab w/ #5@12" O.C. Top and B.E.W. Isolated from adjacent slab.

Blue- Exterior 4" Conc. Slab on grade w/ 6x6W1.4xW1.4 W.W.F sloped away from building.

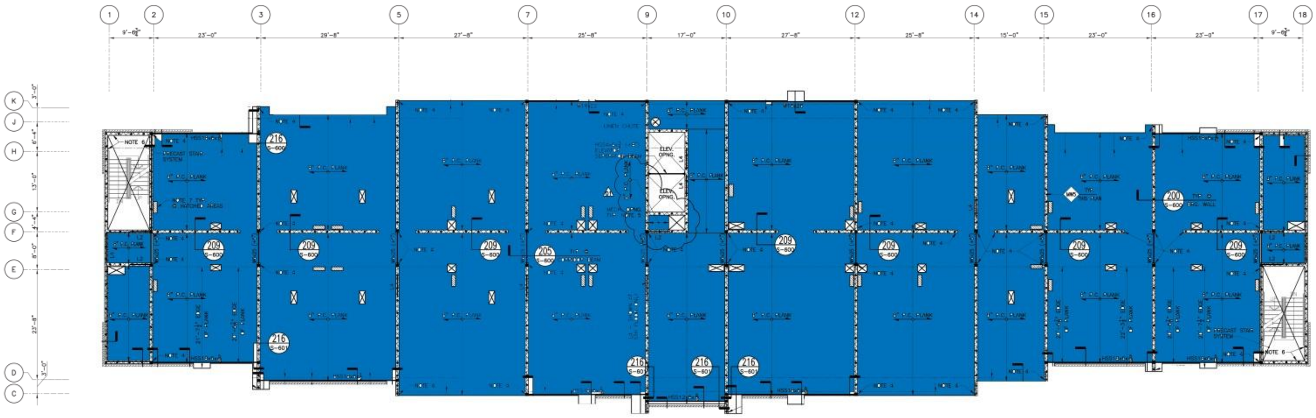


Figure 5: Typical Floor plank layout

Framing System

The framing system for the Hotel N.E.U.S consists of steel columns on the first floor mixed with masonry bearing walls. Due to the gathering areas and general openness of the first floor, steel columns are used. These columns only exist on this floor, save for column C12 and E12 that span the first two floors (see Figure 7) Everywhere else in the building, masonry walls are used to support the floor system. The exterior is supported by cold-formed steel (see Figure 7 for sections) Bays are typical except for on the second floor where an opening exists for an open ceiling breakfast region. The longest bearing wall is about 28' long, located on column line 9 near the center of the building where it is widest.

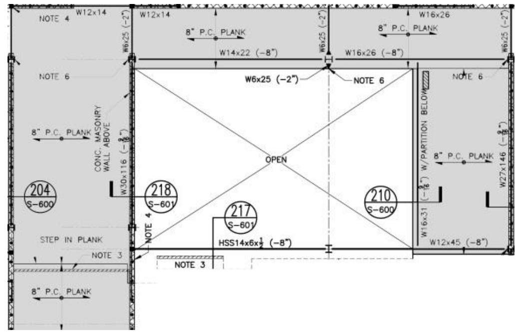
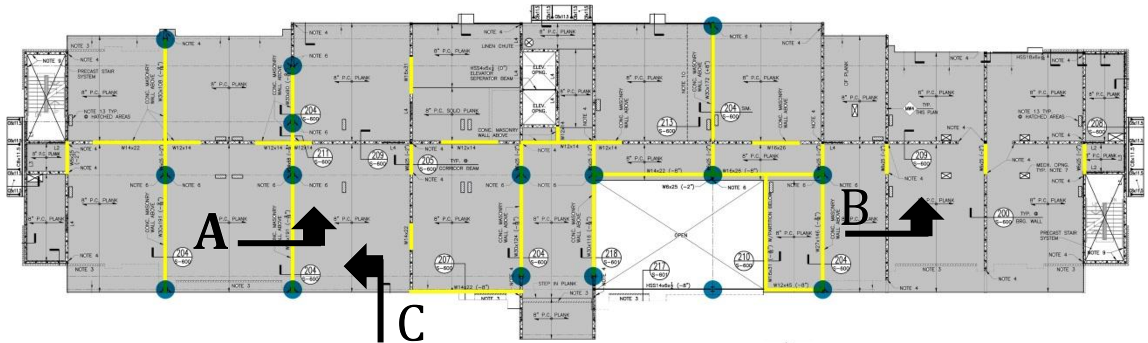
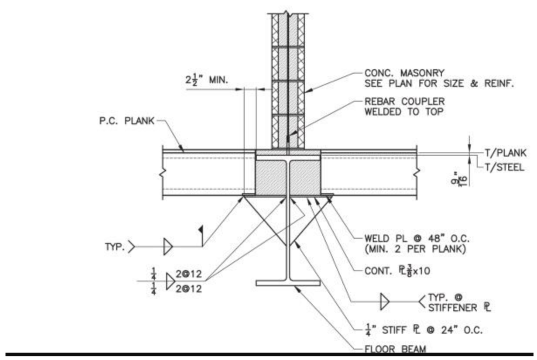
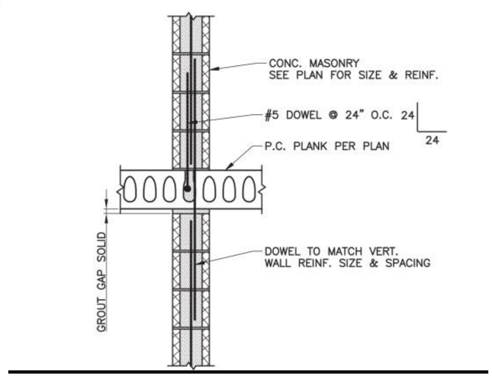


Figure 6: Open section on second floor

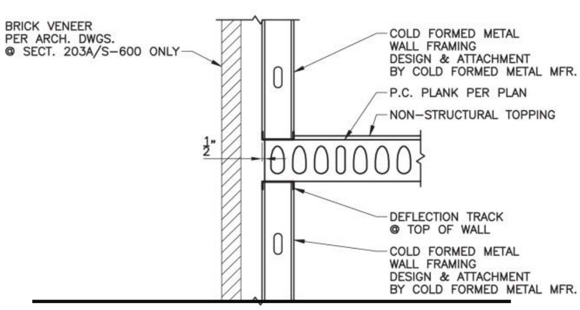




SECTION A- Beam carrying masonry wall



SECTION B- Plank on masonry wall



SECTION C- Plank resting on cold-formed steel at exterior

Figure 7: Second Story framing
Yellow indicates beams
Blue indicates columns

Lateral System

In the Hotel N.E.U.S, the lateral system consists is the same as the gravity system. Reinforced masonry shear walls provide the resistance to lateral loads applied to the building. The masonry is 8" wide with #5 bars at 24" on center. Cells with reinforcement are grouted solid. As with the gravity system, these walls are controlled by the fact that the first floor requires a space without obstructions. Therefore the shear walls are located in an irregular pattern shown in Figure 8. Due to the slenderness of the building, much more resistance is required perpendicular to the long side of the building.

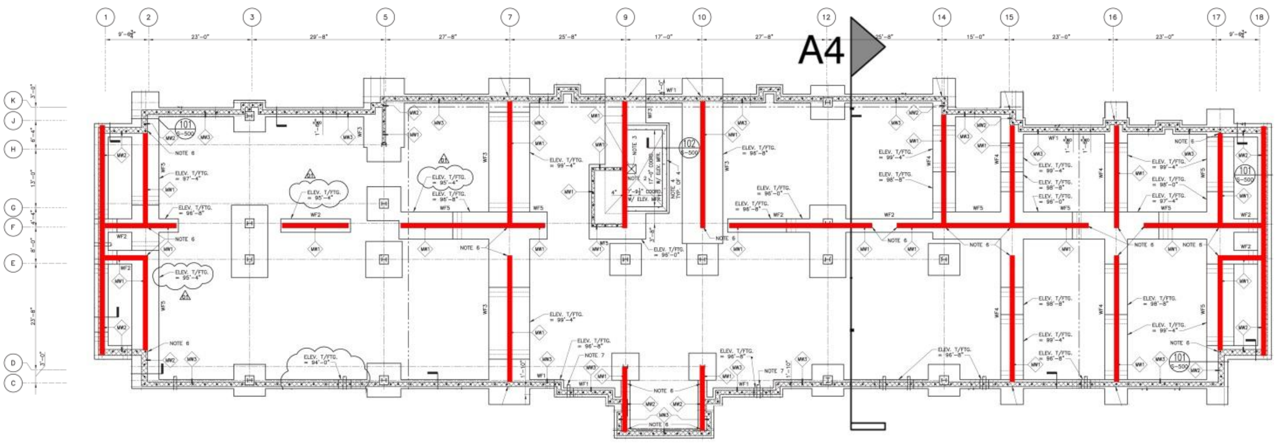


Figure 8: Location of shear walls on foundation plan

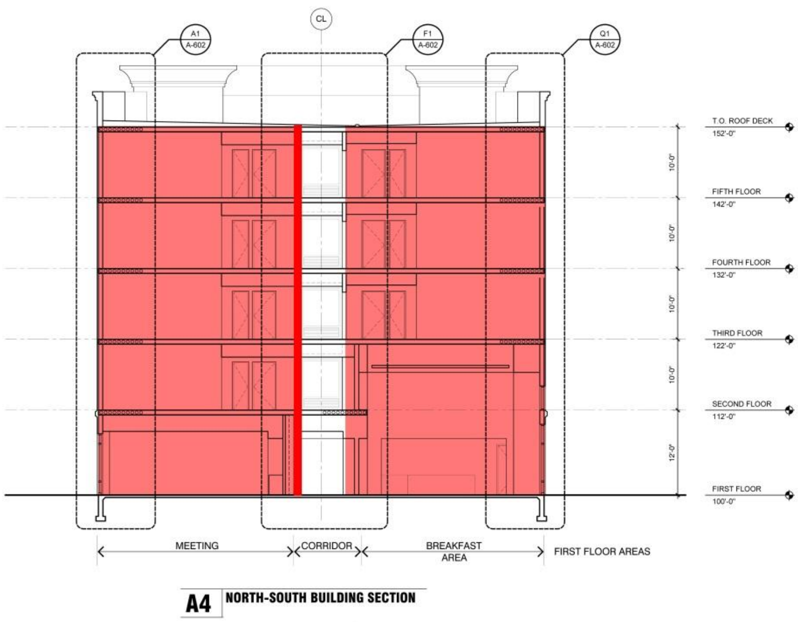
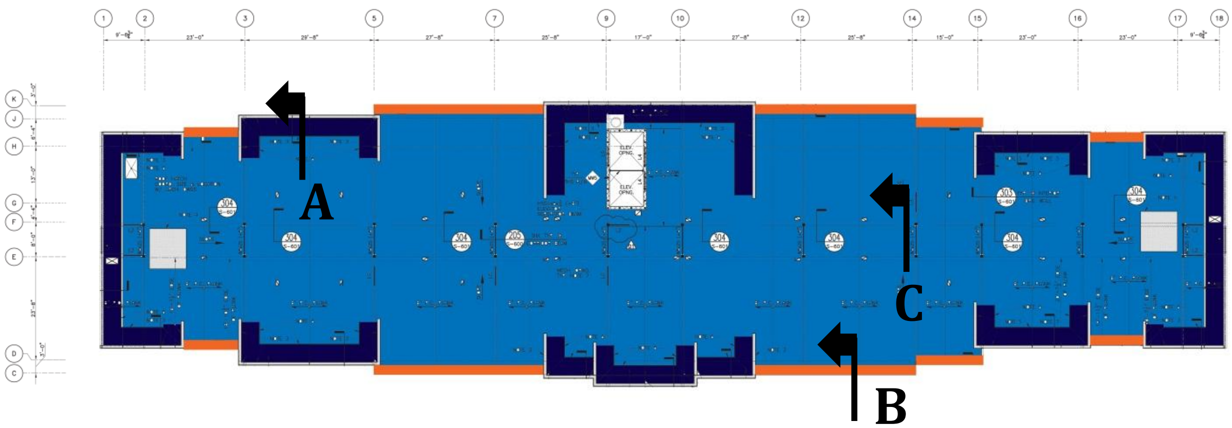
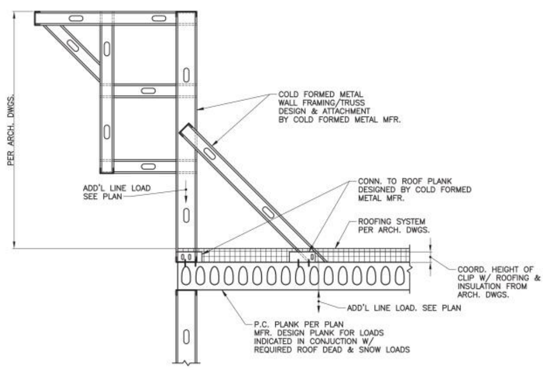


Figure 9: Section showing orientation of shear walls.

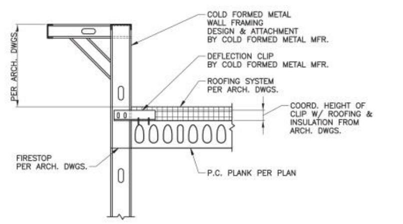
Roof System

As with the floor system, the roof is constructed of 8" Hollowcore Precast plank with insulation on top. A parapet constructed of cold-formed steel engrosses the entire perimeter and is to 8'-8" high. Mechanical units weighing 4,000 lbs each are located at either end of the roof.

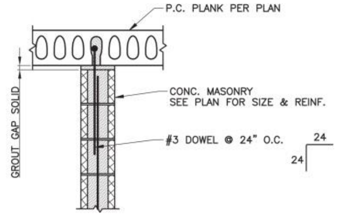




SECTION A- 8'-8" Cold-formed steel parapet wall



SECTION B- 5'-0" Cold-formed steel parapet wall



SECTION C- Roof plank on top of masonry wall

Figure 10: Roof layout.
Blue- 8" Hollowcore Precast Plank
Orange- 5'-0" Cold-formed steel parapet wall
Dark Blue- 8'-8" Cold-formed steel parapet wall

Materials

Listed in Figure 11 are the materials used in the construction of the Hotel N.E.U.S. They were gathered from the structural engineer's general notes and specifications.

Shallow Foundations Wall Footing Capacity	
Width	Allowable Bearing Pressure
2'-0"	4,100 PSF
3'-0"	4,600 PSF
4'-0"	4,500 PSF
5'-0"	3,800 PSF
6'-0"	3,250 PSF
7'-0"	2,800 PSF
8'-0"	2,500 PSF

Column Footing Capacity	
Width	Allowable Bearing Pressure
3'-0"	4,600 PSF
4'-0"	4,500 PSF
5'-0"	3,800 PSF
6'-0"	3,250 PSF
7'-0"	2,800 PSF
8'-0"	2,500 PSF
9'-0"	6,650 PSF
10'-0"	6,250 PSF
11'-0"	5,500 PSF

Reinforced Concrete	
Type	Design Compression Strength (f'c)
Foundations and Concrete Fill	3,000 PSI
Walls	4,000 PSI
Slabs and Grade	4,000 PSI
Reinforcement	
Deformed Bars	ASTM A625 GRADE 60
Deformed Bars (weldable)	ASTM A706, GRADE 60
Welded Wire Fabric	ASTM A185

Figure 11: Material Standards used in Hotel N.E.U.S.

Masonry	
F'm	2,000 PSI
Mortar	ASTM C270
	Type M for all F'm = 2,500 PSI, Type S for all structural masonry
Grout	F'c = F'm but no less than 2,000 PSI

Face Brick
ASTM C216, Grade SW, Type FBS absorption not more than 9% by dry weight per ASTM C67.

Structural Steel	
W shapes	ASTM 992
M, S, C, MC, and L shapes	ASTM A36
HP shapes	ASTM A572, GRADE 50
Steel Tubes (HSS shapes)	ASTM A500, GRADE B
Steel Pipe (Round HSS)	ASTM A500, GRADE B
Plates and Bars	ASTM A36
Bolts	ASTM A325, TYPE 1, 3/4" U.N.O.

Galvanized Structural Steel	
Structural Shapes and Rods	ASTM A123

Precast Concrete	
Type	Design Compression Strength (f'c)
Reinforcement (deformed)	ASTM A 615/A 615M, Grade 60
Welded Wire Reinforcement:	ASTM A 185
Pretensioning Strand	ASTM A 416/A 416M, Grade 250 or Grade 270, uncoated, 7-wire, low-relaxation strand wire or ASTM A 886/A 886M, Grade 270, indented, 7-wire, low-relaxation strand
Portland Cement	ASTM C 150

Figure 12: Material Standards used in Hotel N.E.U.S.

Design Codes

Because of the wide variety of materials used on this project there are also many different codes to abide by. These are listed in Figure 13. The codes used for analysis in this thesis are listed in Figure 14. For a list of other codes used see Appendix A.

Structural Design Codes	
Reinforced Concrete	Building Code Requirements for Structural Concrete (ACI 318, latest)
	Specifications for Structural Concrete (ACI 301, latest)
Masonry	Building Code Requirements for Masonry Structures (ACI 530)
	Specifictations for Masonry Structures (ACI 530.1)
Precast Concrete	Building Code Requirements for Structural Concrete (ACI 318, latest)
	Commentary (ACI 318R, latest)
	PCI Design Handbook - Precast and Prestressed Concrete (PCI MNL 120)
Structural Steel	Specification for Structural Steel Buildings (ANSI/AISC 360-05)
Metal Decking	Steel Roof Deck Specifications and Load Tables (Steel Deck Institute, latest edition)
Cold Formed Steel	Most current edition of the "North Amercian Specification for the Design of Cold-Formed Steel Framing"
Wind and Seismic	ASCE 7-05
Loads	International Building Code 2009

Figure 13: Codes used by the engineer of record to design this structure

Thesis Analysis Codes	
Reinforced Concrete	Building Code Requirements for Structural Concrete (ACI 318-11)
Precast Concrete	PCI Design Handbook - Precast and Prestressed Concrete (PCI MNL 120)
Structural Steel	AISC Steel Manual 14th Edition
Wind and Seismic	ASCE 7-05
Loads	International Building Code 2009

Figure 14: Codes used for thesis

Gravity Loads

The dead loads for this structure were either provided by the engineer of record or assumed by referencing structural handbooks. The plank weight was obtained using PCI Manual 120 and Masonry walls were determined using NCMA TEK 14-13B. The density was assumed as 105 lb/ft^3 as it was described as "medium" in the specifications. The topping is to level the surface since the camber of the plank will cause it to be uneven. These loads prove to be very similar to the overall load used by the engineer of record as the spot checks performed give good results.

Dead Loads	
Location	Load (psf)
8" Precast Plank	56
3/4" Topping	6
MEP/Misc.	5
Ceiling	3
Roof Insulation	12
C.F. Studs	5
Roof	20
Masonry Walls	43-53

Figure 15: Dead Loads for Hotel N.E.U.S.

Live loads were listed in the general notes no sheet S001. All of them were in accordance with the International Building Code 2009. Due to the typical layout of floors in a hotel, 40 psf was used on the entire floor except for stairwells on floors two through five. The engineer of record used live load reduction when determining loads for the beams, columns, and column footings. However, there was no reduction for the wall footing.

Live Loads			
Location	Design Live Load (psf)	IBC 2009 Live Load (psf)	Reference Note
Public Areas	100	100	Residential - hotels and multifamily dwellings - public rooms and corridors serving them
Guest Rooms and Corridors	40	40	Residential - hotels and multifamily dwellings - private rooms and corridors serving them
Paritions	20	20	
Stairs	100	100	Stairs and exits - all other
Roof	20	20	Roofs - ordinary flat, pitched, and curved roofs

Figure 16: Live Load comparison and references

Snow Loads

The seventh chapter of ASCE 7-05 was used to determine the snow loads on the roof of Hotel N.E.U.S. A ground snow load of 30 psf was used to be conservative (instead of the 25 psf from Figure 7-1). The Exposure factor was also taken conservatively at a value of 1.0. Thermal and Importance factors were determined to be 1.0 as well. Using the equation 7-1 the flat roof snow load was 21 psf and is used as the base value across the entire roof.

Due to the parapet surround the entire roof, drift is a major concern. Using chapter 7.7.1 and 7.8, the snow drift was calculated. A parapet only allows for windward drift to occur, therefore the length used is the upwind distance and the drift height is reduced by a factor of 0.75.

The design engineer used an area load of 42 psf for snow, which is 2/3 of the load they calculated. Snow loads are interpreted differently per engineer and this was one way of signifying how it was designed. The snow drift in the short direction here is very close to what is used in the actual design.

Flat Roof Snow Load			
Factors			Reference
Ground Snow Load	p_g	30 psf	Figure 7-1
Importance Factor	I	1.0	Table 7-4
Exposure Factor	C_e	1.0	Table 7-2
Thermal Factor	C_t	1.0	Table 7-3
Flat Roof Snow Load	p_f	21 psf	Eq. 7-1
Snow Drift (short)			
Factors			Reference
Specific Gravity	γ	17.9 pcf	Eq. 7-3
Upwind Length	l_u	60'	
Base Accumulation	h_b	1.17'	7.7.1 and 7.8
Base to Top of Parapet	h_c	7.5	
Height of Drift	h_d	2.05'	
Width of Drift	w	8.2'	
Peak Drift Load	p_d	57 psf	
Snow Drift (long)			
Factors			Reference
Specific Gravity	γ	17.9 pcf	Eq. 7-3
Upwind Length	l_u	258'	
Base Accumulation	h_b	1.17'	7.7.1 and 7.8
Base to Top of Parapet	h_c	7.5	
Height of Drift	h_d	4.0	
Width of Drift	w	16.12'	
Peak Drift Load	p_d	72.28 psf	

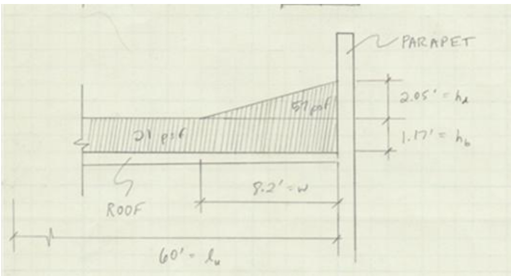


Figure 17: Flat snow and drift loads

Rain Loads

Typically a roof has a main drain on the roof and secondary drains that are further up the slope in case there is a backup. This roof system has scuppers along the base of the parapets instead of secondary drains. The International Building Code 2009 with Commentary provides charts and equations to calculate the amount of rain if there were to be a full backup of the drains. Figure _ was developed from IBC 2009 and used to compute the load for this building. The scuppers on the Hotel N.E.U.S are sized at 8"x6", therefore a 6"x6" was used to be conservative. A total vertical distance of 9" spans between the bottom of the scupper and the main drain. Using a tributary area based off the roof plan and drain locations, it was determined that a 2" hydraulic head can accumulate on top of a 9" static head above the main drain. A load of 57.2 psf would exist at this location, which is nearly the same as the roof drift along the side of the building. These loads would not exist at the same time, but by comparing them it shows the significance of having properly placed drainage.

Drainage System		8	
Static Head	d_s	9	in
Tributary Area	A	1750	ft2
Rainfall Rate	*i*	2.75	in/hr
	i	0.229167	ft/hr
Flow Rate	Q	49.99653	gpm
Hydraulic Head	d_h	2	in
Rain Load	R	57.2	psf

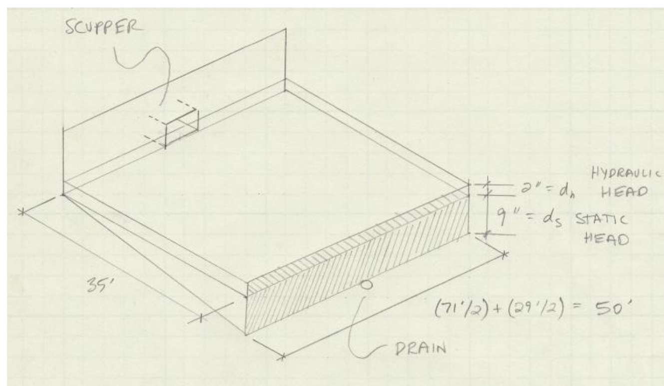


Figure 18: Rain Load variables

#	Drainage System	Flow Rate (gpm) Hydraulic head: 1	2	2.5	3	3.5	4	4.5	5	7	8
1	4" dia	80	170	180							
2	6" dia	100	190	270	380	540					
3	8" dia	125	230	340	560	850	1100	1170			
4	6" open top	18	50		90		140		194	321	393
5	24" open top	72	200		360		560		776	1284	1572
6	6x4	18	50		90		140		177	231	253
7	24x4	72	200		360		560		8	924	1012
8	6x6	18	50		90		140		194	303	343
9	24x6	72	200		360		560		776	1212	1372

See IBC 2009 w/ Commentary, §1661 pg. 16-63

Figure 19: Reference Chart based off of IBC 2009

Beam Check

This beam, a W30x191 located along column line 5 between C and E is 26'-8" long. It was selected because it holds four stories of masonry bearing wall above. The beam was adequate to carry the gravity loads, but used 90% of its capacity. The live load deflection was 0.274" which was about 30% of the maximum allowable by code. This seems low, but the majority of the load this beam is supporting is the plank and wall of the stories above. The total load deflection was 0.64" which is half of the allowable 1.33". This is the controlling deflection limit case, but the beam's overall strength is really the limiting factor.

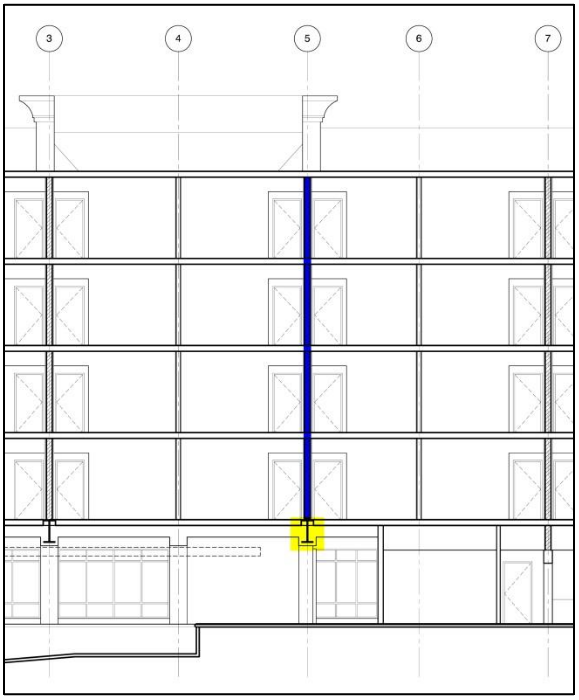


Figure 20: Section through beam

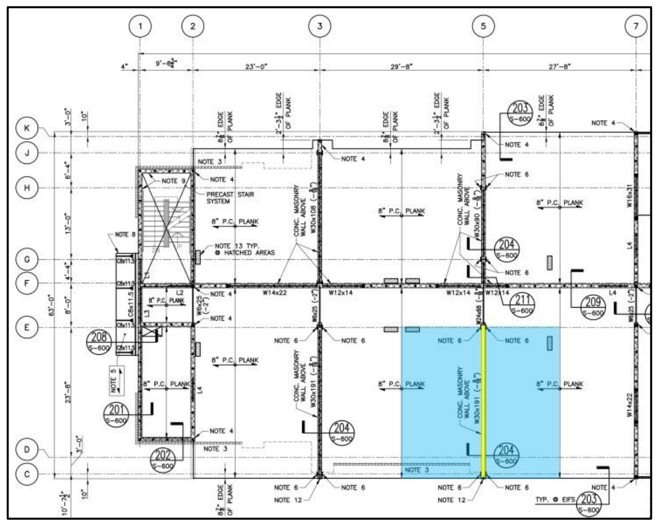


Figure 21: Tributary area of beam

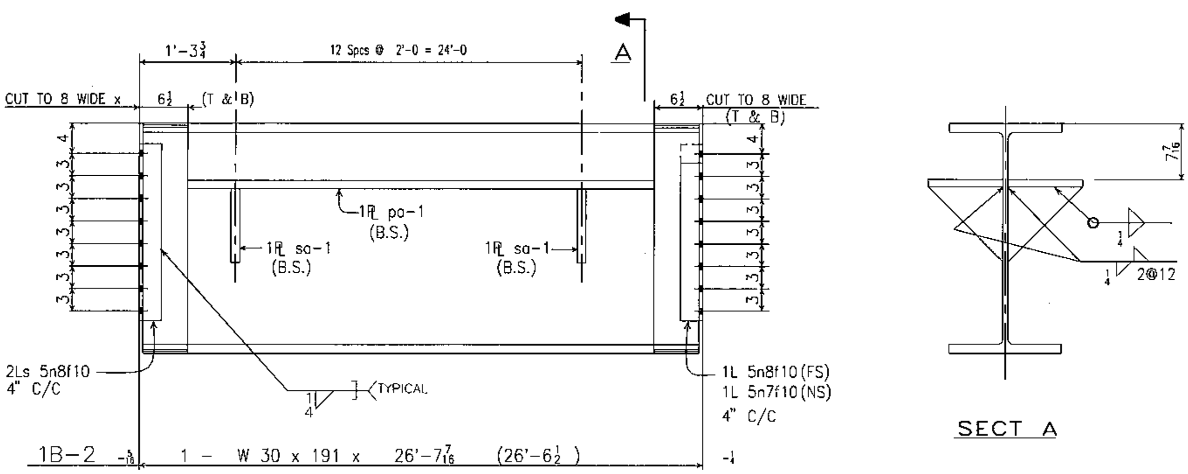


Figure 22: Shop drawing of beam

Column Check

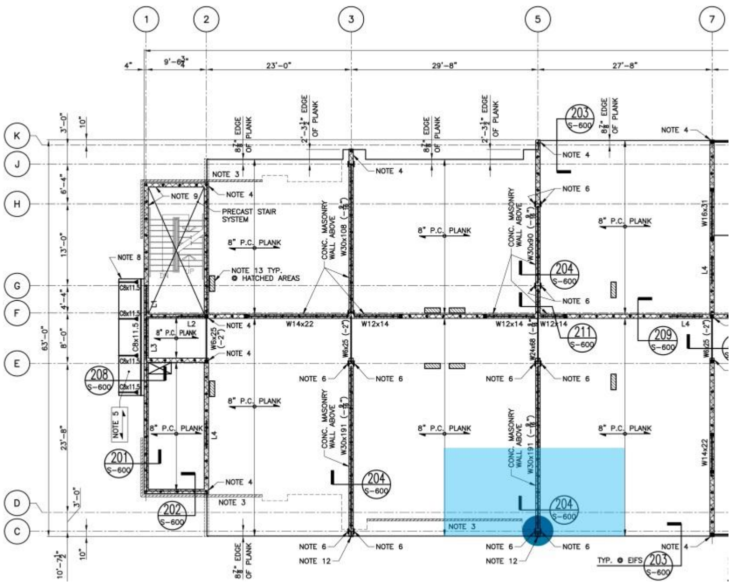


Figure 23: Tributary area of column

Column C5 was selected because it supports the beam that was checked previously. Also, since it is an exterior column, the façade's weight must be carried by it as well. It is a W12x96 with a 1.5"x19"x19" baseplate. The footing is 7'-6"x7'x6"x1'-8" with 8-#7 bars. A 24"x24" pier is spans between the baseplate and footing.

A load of 293 kips to the baseplate was calculated using the dead and live loads shown in Figure 15 and Figure 16. The column schedule on sheet S400 has the load to the baseplate listed as 295 kips. This serves as a confirmation that the loading used is accurate. The column was checked for weak axis buckling for gravity loads. It was found to use about 26% of its capacity and is adequate to support the structure above.

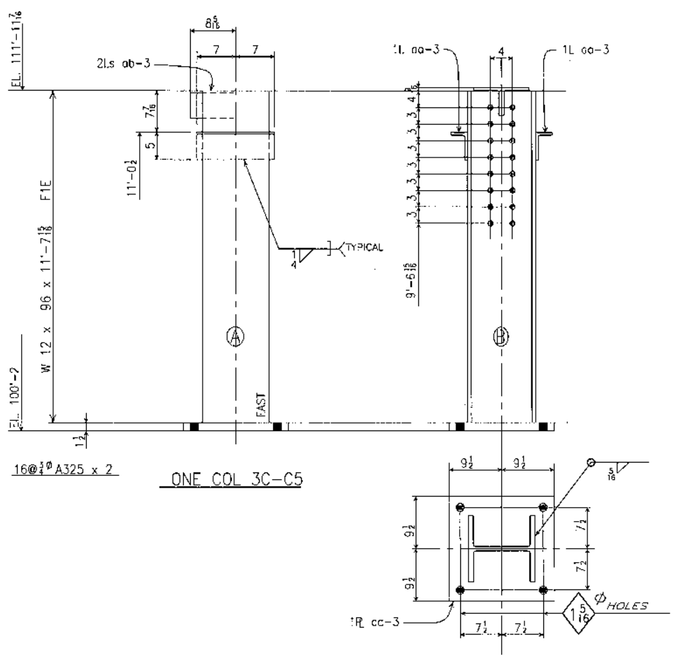


Figure 24: Column shop drawing

Floor Check

A prestressed analysis was used to determine whether the plank used in Guestroom 223. The planks are 8” Hollowcore precast concrete with prestressed strands and is 25’-8” long. The values used in this check were obtained from the PCI Manual 120-04. These values may differ slightly from those of the manufacturers listed in the specifications.

A plank with 6 strands at 6/16” was found to be overprestressed for the loads it has to carry. The reason for performing this analysis was to understand the effects of the prestressed strands. However in practice, many engineers will use the load tables to save time on projects. In Figure 24 you can see the table of safe loads and highlighted is the span of the plank in Guestroom 223. A total of 130 psf exists on the plank, thus a 48-S plank can be used to satisfy the capacity requirements.

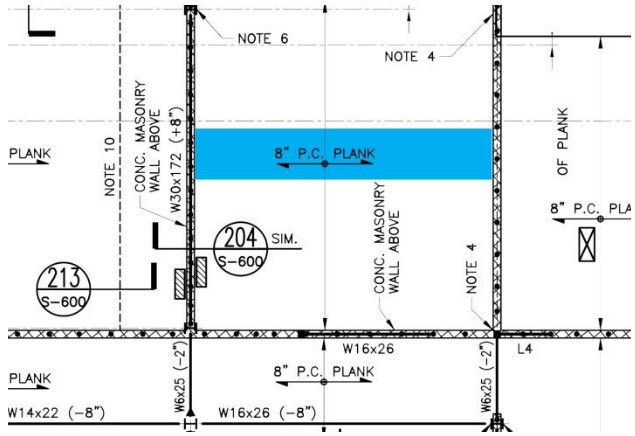


Figure 25: Area of one plank

Strand Pattern Designation
48-S

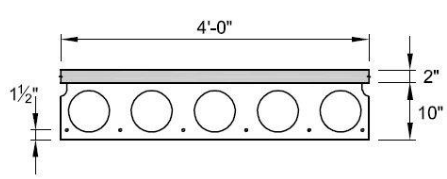
- S = straight
- Diameter of strand in 16ths
- No. of Strand (4)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
258 – Safe superimposed service load, psf
0.3 – Estimated camber at erection, in.
0.4 – Estimated long-time camber, in.

HOLLOW-CORE
4’-0” x 10”
Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi

Section Properties

	Untopped		Topped	
A =	259	in.^{2}	355	in.^{2}
I =	3,223	in.^{4}	5,328	in.^{4}
y_b =	5.00	in.	6.34	in.
y_t =	5.00	in.	5.66	in.
S_b =	645	in.^{3}	840	in.^{3}
S_t =	645	in.^{3}	941	in.^{3}
wt =	270	plf	370	plf
DL =	68	psf	93	psf
V/S =	2.23	in.		

4HC10

Table of safe superimposed service load (psf) and cambers (in.) — **No Topping**

Strand Designation Code	Span, ft																									
	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46
48-S	258	234	209	187	168	151	136	123	111	100	90	82	74	66	60	54	48	43	38	34	30	26					
	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.4	–0.6	–0.7	–0.9					
	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.8	–1.1	–1.3	–1.3	–1.9					
58-S	267	249	237	223	211	197	179	162	148	134	122	112	102	93	85	77	70	64	58	53	48	43	39	35	30	26	
	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.4	0.4	0.3	0.2	0.2	0.1	0.0	–0.1	–0.3	–0.4	–0.6	–0.7	–0.9	–1.2	
	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	0.0	–0.1	–0.3	–0.5	–0.7	–1.0	–1.2	–1.5	–1.8	–2.2	–2.6	
68-S	273	255	243	229	217	206	196	187	176	162	153	141	129	118	109	100	92	84	78	71	65	60	54	49	44	39	34
	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	–0.1	–0.2	–0.4	–0.6	–0.8
	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.9	0.9	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	–0.3	–0.6	–0.8	–1.1	–1.4	–1.8	–2.2
78-S	282	264	249	235	223	212	202	193	185	174	165	153	144	136	129	119	113	104	96	89	82	76	69	63	57	52	47
	0.6	0.7	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	0.9	1.0	1.0	1.0	0.9	0.9	0.9	0.8	0.8	0.7	0.6	0.5	0.4	0.3	0.1	0.0	–0.2
	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	1.0	0.9	0.8	0.6	0.5	0.3	0.1	–0.1	–0.4	–0.7	–1.0	–1.3
88-S	288	270	255	241	229	218	208	199	188	180	174	165	153	145	135	128	122	115	106	101	96	91	84	77	71	65	59
	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.1	1.1	1.0	0.9	0.8	0.7	0.5	0.3
	1.0	1.0	1.1	1.2	1.2	1.3	1.3	1.4	1.4	1.4	1.4	1.5	1.5	1.4	1.4	1.4	1.3	1.2	1.2	1.0	0.9	0.7	0.6	0.3	0.1	–0.2	–0.5

Figure 26: PCI load table

Lateral Loads

To gain a deeper understanding the lateral forces in the Hotel N.E.U.S., wind and seismic loads were calculated by using ASCE 7-05. The shear wall system is designed to resist these loads and will be examined in further reports.

Wind Analysis

The wind loads for the Main Wind Force Resisting System were calculated by the analytical procedure outlined in chapter 6 of ASCE 7-05. The building was simplified into a rectangle that was 258' x 61'. The tallest parapet height of 60'-8" was assumed to encompass the entire perimeter. Although the footprint of the building sits at an angle, the North-South direction is associated with the longer face of the building while East-West is the short sides.

Hotel N.E.U.S. was determined to be an occupancy category II with an importance factor of 1. The exposure category was C and the topographic factor was 1 as well. Since this the Hotel is a rigid building (which was determined by having a period 1< in the seismic section), the gust factor was calculated for each direction. The values acquired were 0.8386 and 0.872 for NS and EW respectively. To be conservative, a factor of 0.85 was used for the continuation of the analysis.

The parapet pressures were designed in accordance with 6.5.11.5, where a factor of 1.5 is used for windward parapets and -1.0 for leeward parapets. The force associated with these pressures should be used in the design of the MWRFS. However, components and cladding wind loads should be used in the design of the parapet itself.

It was determined that the overturning moment in the North-South direction was greater than four times that of the East-West direction. This is a result of the large difference in surface area from side to side. Figure _ shows all factors and coefficients used in the calculations. In Figure _ the velocity pressures are shown. Pressures and forces calculated for design are listed in Figure 30, 31, 33, and 34.

Wind Load Data		
Design Wind Speed	V	90
Directionality Factor	Kd	0.85
Occupancy Category	I	II
Importance Factor		1
Exposure Category		C
Topographic Factor	Kzt	1
Internal Pressure Coefficient	Gcpi	+/-0.18
Gust Factor	G	.85

Figure 27: Factors and Coefficients

Wall Pressure Coefficients		
Windward	Cp	0.8
Side Wall (N-S)	Cp	-0.5
Side Wall (E-W)	Cp	-0.2
Leeward	Cp	-0.7
Roof Pressure Coefficients		
Windward (E-W)	0-h/2	-0.9
	h/2-h	-0.9
	h-2h	-0.5
	>2h	-0.3
Windward (N-S)	0-h/2	-1.3
	>h/2	-0.56

Figure 28: Coefficients

Velocity Pressures							
Level	Elevation	K_z	K_{zt}	K_d	V^2	I	q_z
	60.67	1.1327	1	0.85	8100	1	19.964
Parapet	52	1.098	1	0.85	8100	1	19.3529
5	42	1.05	1	0.85	8100	1	18.5069
4	32	0.992	1	0.85	8100	1	17.4846
3	22	0.916	1	0.85	8100	1	16.145
2	12	0.85	1	0.85	8100	1	14.9818
Ground	0	0.85	1	0.85	8100	1	14.9818

Figure 29: Velocity Pressures

Wind Pressures N-S								
Location	Level	Distance (ft)	Velocity Pressure (psf)	External Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
			q_p / q_z / q_h	p_p / p_z / p_h (psf)	Positive (GCp)	Negative (GCp)	Positive	Negative
Windward		60.67	19.96	29.95	1.5		29.95	
	Parapet	52	19.35	13.16	2.70	-2.70	15.86	10.46
	5	42	18.51	12.58	2.70	-2.70	15.28	9.89
	4	32	17.48	11.89	2.70	-2.70	14.59	9.19
	3	22	16.15	10.98	2.70	-2.70	13.68	8.28
	2	12	14.98	10.19	2.70	-2.70	12.88	7.49
	Ground	0	14.98	10.19	2.70	-2.70	12.88	7.49
Leeward	Parapet	60.67	19.96	-19.96	-1.0		-19.96	
	G-4	52	14.98	-8.91	2.70	-2.70	-6.22	-11.61
Side	All	Total	14.98	-2.55	2.70	-2.70	0.15	-5.24
Roof	-	0-30.33	14.98	-11.46	2.70	-2.70	-8.76	-14.16
	-	30.33-60.67	14.98	-11.46	2.70	-2.70	-8.76	-14.16
	-	60.67-121.33	14.98	-6.37	2.70	-2.70	-3.67	-9.06
	-	>121.33	14.98	-3.82	2.70	-2.70	-1.12	-6.52

Figure 30: Wind Pressures N-S

Wind Forces N-S						
Level	Elevation (ft)	Tributary Area (ft^2)		Wind Force (k)	Story Shear (k)	Overturning Moment (ft-k)
		Above	Below			
	60.67	0	1118	55.82	55.82	3386.64
Parapet	52	1118	1290	84.29	140.12	4383.34
5	42	1290	1290	56.21	196.32	2360.67
4	32	1290	1290	54.57	250.89	1746.16
3	22	1290	1290	52.50	303.39	1154.91
2	12	1290	1548	55.23	358.61	662.74
Ground	0	1548	0	0.00	358.61	0.00
						13694.46

Figure 31: Wind Forces, Story Shear, Overturning Moment N-S

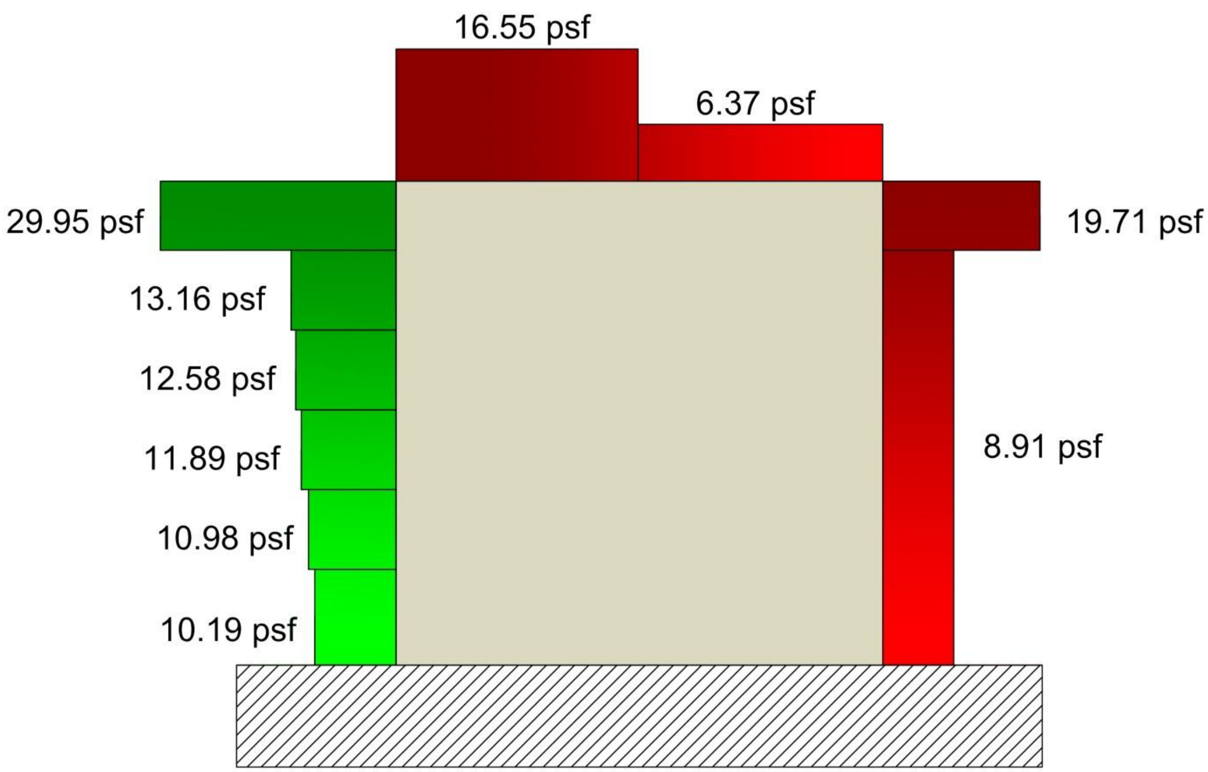


Figure 32: Diagram of Pressures N-S

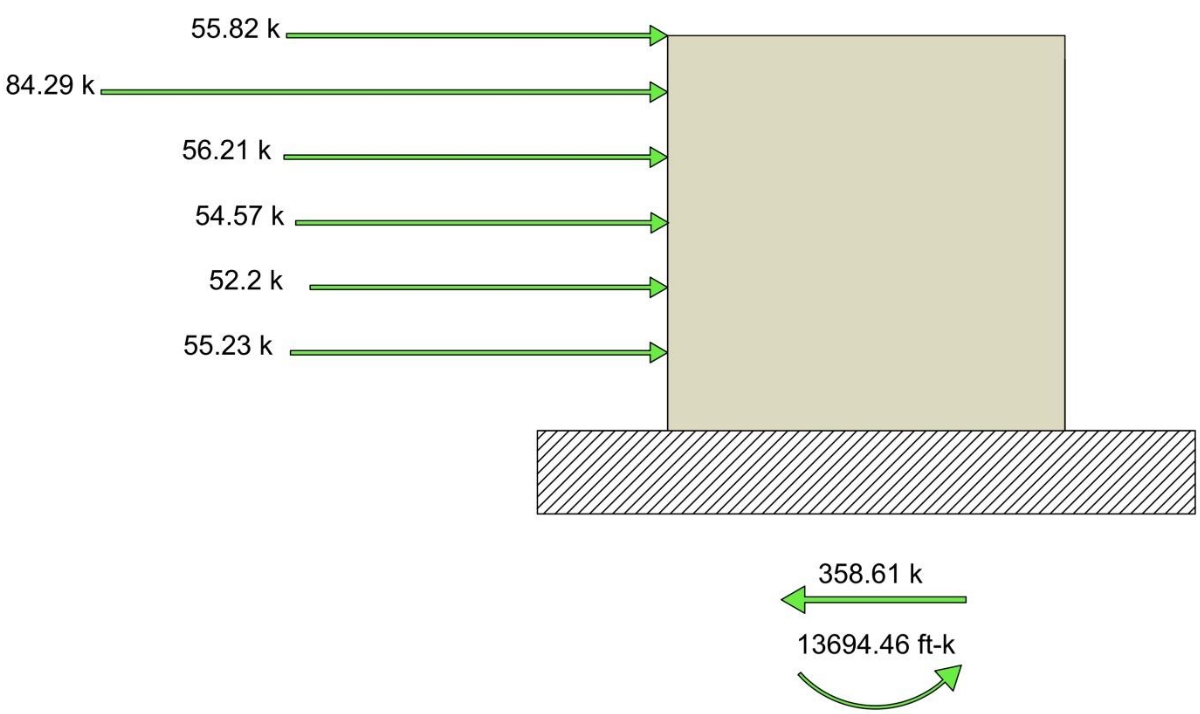


Figure 33: Diagram of Forces N-S

Wind Pressures E-W								
Location	Level	Distance (ft)	Velocity Pressure (psf)	External Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
			q_p / q_z / q_h	p_p / p_z / p_h (psf)	Positive (GCp)	Negative (GCp)	Positive	Negative
Windward	Parapet	60.67	19.96	29.95	1.50		29.95	
	5	52	19.35	13.16	2.70	-2.70	15.86	10.46
	4	42	18.51	12.58	2.70	-2.70	15.28	9.89
	3	32	17.48	11.89	2.70	-2.70	14.59	9.19
	2	22	16.15	10.98	2.70	-2.70	13.68	8.28
	Ground	12	14.98	10.19	2.70	-2.70	12.88	7.49
	Base	0	14.98	10.19	2.70	-2.70	12.88	7.49
Leeward	Parapet	60.67	19.96	-19.96	-1.0		-19.96	
	G-4	52	14.98	-8.91	2.70	-2.70	-6.22	-11.61
Side	All	Total	14.98	-6.37	2.70	-2.70	-3.67	-9.06
Roof	-	0-28.5	14.98	-16.55	2.70	-2.70	-13.86	-19.25
	-	>h/2	14.98	-7.13	2.70	-2.70	-4.43	-9.83

Figure 33: Wind Pressures E-W

Wind Forces E-W						
Level	Elevation (ft)	Tributary Area (ft^2)		Wind Force (k)	Story Shear (k)	Overturning Moment (ft-k)
		Above	Below			
	60.67	0	264	13.20	13.20	800.72
Parapet	52	264	305	19.93	33.13	1036.37
5	42	305	305	13.29	46.42	558.14
4	32	305	305	12.90	59.32	412.85
3	22	305	305	12.41	71.73	273.06
2	12	305	366	13.06	84.79	156.69
Ground	0	366	0	0.00	84.79	0.00
						3237.84

Figure 34: Wind Forces, Story Shear, Overturning Moment E-W

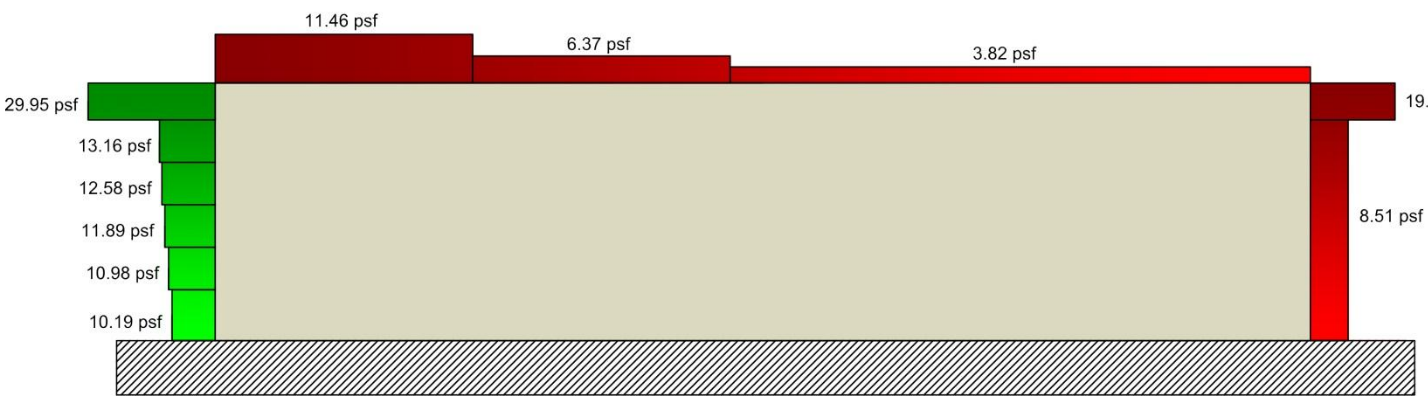


Figure 35: Diagram of Pressures E-W

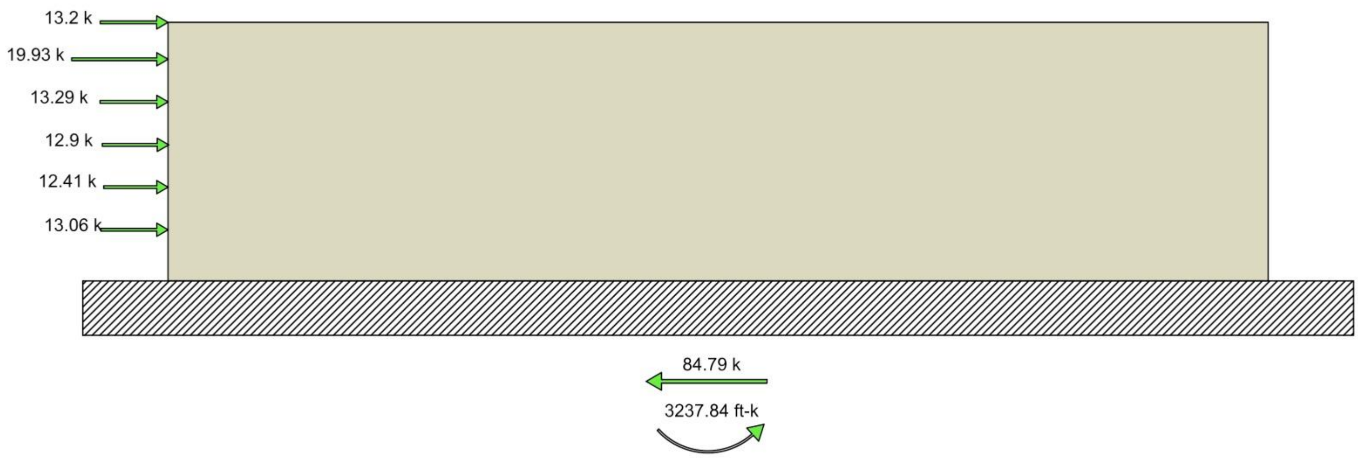


Figure 36: Diagram of Forces E-W

Seismic Analysis

The Equivalent Lateral Force procedure outlined in ASCE 7-05 is used to calculate the seismic loads. The fundamental frequency was calculated for both the general equation (12.8-7) and for masonry shear walls (12.8-9). A Response Modification Coefficient of 2 was used for the a system designated as Reinforced Masonry Shear Walls. The Hotel N.E.U.S. doesn't fit in category but "Other Structures" for the general equation of frequency. The values for the N-S and E-W direction by equation 12.8-9 are much less and can be seen in Appendix D. This could likely be due to the estimates in the length of each shear wall and base area. The general equation was used in this analysis so base shear could be compared to that of the design engineer's value. As was stated in the wind analysis, this structure has a fundamental period that is less than one, classifying it as rigid.

The engineer of record used a coefficient of 0.67 which is from equation 12.8-2. However, by equation 12.8-3, when T is less than T_L, the value of C_s has a maximum limited by the period. A value of 0.06 was found as the allowed max for the building and is used with the weight calculated (see Appendix D). A base shear of 637 kips was about 56 kips off of the engineer of record's value on sheet S001. A 10% difference in values shows that the factors and weights used in this analysis were fairly accurate for a hand calculated base shear. The design engineer used RAM Structural to obtain these values. This is much more accurate in determining the seismic weight. The overturning moment is 25,440 foot kips and is much larger than the overturning moment due to wind. Wind generally controls in this region of the United States, but being constructed of masonry and plank, this building is very heavy which results in this larger value.

Seismic Load Data		
Occupancy Category	-	II
Site Class	-	D
Seismic Load Importance Factor	I_e	1
Site Class Coefficient	S_s	0.125
	S_1	0.049
Spectral Response Coefficient	F_a	1.6
	F_v	2.4
	S_{DS}	0.1333
	S_{D1}	0.0784
Seismic Design Category	-	B
Response Modification Factor	R	2
Long Period Transition Period	T_L	12
Fundamental Period	Ta	0.387

Figure 37: Seismic Data

Total Building Weight				
Level	Area (ft^2)	Load (k)	Wall Weight (k)	Total (k)
Ground	15725	0	352.13	352.13
2	13133	1051	1575.91	2626.55
3	14370	1150	1443.37	2592.97
4	14370	1150	1442.33	2591.93
5	14370	1092	1442.33	2534.45
			Total Weight(k)	**10698.03**

Figure 39: Weight Calculation

Masonry Shear Wall Data (Cw) for E-W							
Type	C_u	T_a	T	C_{smin}		C_{smax}	C_s
E-W	1.7	0.122	0.207	0.012	0.010	**0.189**	**0.067**
N-S	1.7	0.080	0.136	0.012	0.010	**0.288**	**0.067**
General	1.7	0.387	0.658	0.012	0.010	**0.060**	**0.067**

Base Shear					
Type	Weight	C_s	V (k)	C_s	V (k)
E-W	10698.0	0.067	**717**	0.067	**717**
N-S	10698.0	0.067	**717**	0.067	**717**
General	10698.0	0.067	**717**	0.060	**637**

Figure 38: Base Shear Calculations

Vertical Force Distribution								
Level	Weight (k) w_x	Height (ft) h_x	k	$w_x h_x^k$	Distribution Factor C_{vx}	Story Force (k) $F_x=C_{vx}V$	Story Shear (k)	Overturning Moment (ft-k)
5	2534.45	52	1	131791.40	0.34	217.68	217.68	11319.31
4	2591.93	42	1	108861.06	0.28	179.81	397.48	7551.82
3	2592.97	32	1	82975.04	0.22	137.05	534.53	4385.58
2	2626.55	22	1	57784.10	0.15	95.44	629.98	2099.72
Ground	352.13	12	1	4225.61	0.01	6.98	636.95	83.75
				385637.21	1.00			25440.18

Figure 40: Force Distribution

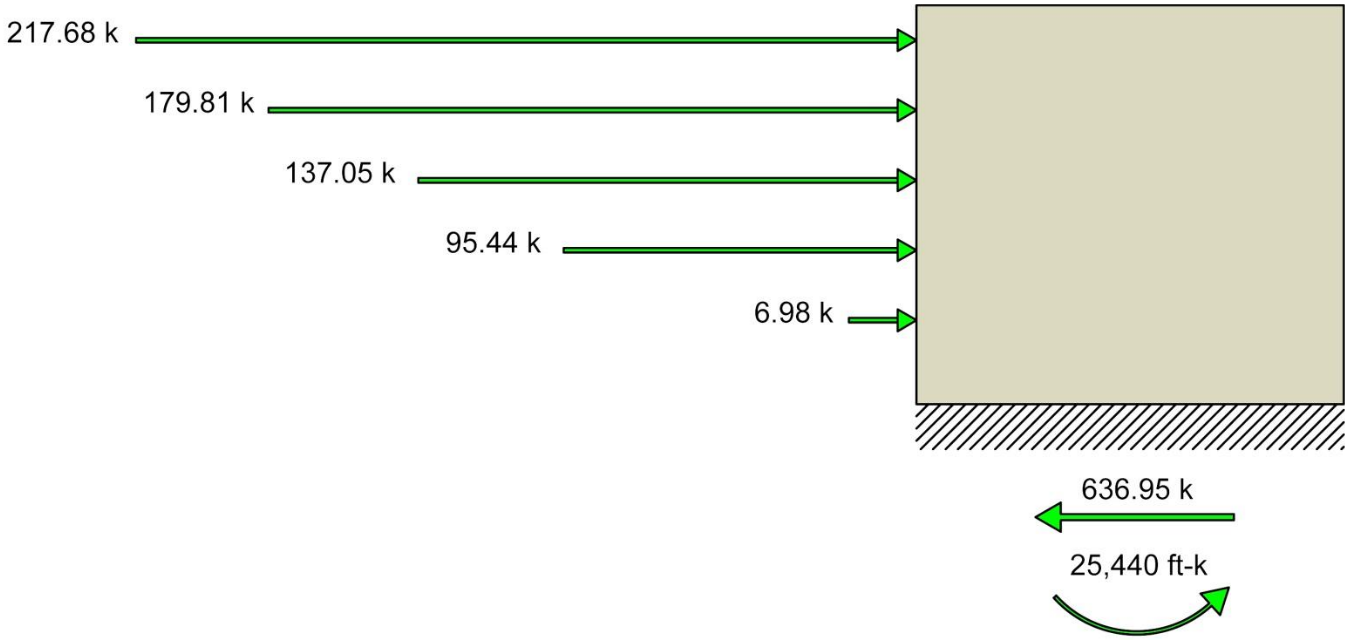


Figure 41: Diagram of Forces. Note: these values are the same for both directions since the general equation for fundamental frequency was used.

Conclusion

Technical Report 1 proved to be a thorough investigation and breakdown of the existing conditions of the Hotel N.E.U.S. From the foundations to the floor, to the frame and lateral system, and lastly the roof, the makeup and identity of this structure was dissected and evaluated. The foundations had varying bearing pressures dependent on size, but foundations in the middle of the buildings were able to be reduced due to existing concrete below. A masonry bearing/shear wall system with precast plank presented itself as a great answer to the spans up to 30' and a regular layout for most floors. Steel beams and columns were used on the first floor to allow for open space as is needed for a hotel establishment.

By referencing ASCE 7-05 and IBC 2009, the gravity loads and weight of the building were determined. Comparisons between thesis and design loads prove to be very similar which was expected since the building is relatively simplistic. Spot checks performed resulted in positive results for all three cases. The plank, beam, and column were adequate to carry their respective loads.

Wind and Seismic loads were explored in depth. The large parapet was found to be a significant factor in the load to the MWFRS. Since the Hotel is so narrow, one direction had forces over four times larger than the other. In seismic, the weight of the building was determined and was most likely overestimated due to constraints of calculating it by hand. There was a base shear difference of 10% between thesis analysis and design values. This can be attributed to the engineer of record using a computer model to obtain data.

Appendices

Appendix A: Plans, Sections, Schedules

MISCELLANEOUS LINTEL SCHEDULE				
WALL THICKNESS	MASONRY OPNG. UP TO 4'–0"	MASONRY OPNG. 4'–0" TO 6'–0"	MASONRY OPNG. 6'–0" TO 8'–0"	MASONRY OPNG. 8'–0" TO 13'–0"
4" WALL	L3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{5}{16}$	L4x3$\frac{1}{2}$x$\frac{5}{16}$	L5x3$\frac{1}{2}$x$\frac{5}{16}$	C7x8.9 + PL$\frac{5}{16}$x5$\frac{1}{2}$
6" WALL	⅃L3$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{5}{16}$	⅃L3$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{5}{16}$	⅃L3$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{3}{8}$	----
8" WALL	⅃L3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{5}{16}$	⅃L4x3$\frac{1}{2}$x$\frac{5}{16}$	⅃L5x3$\frac{1}{2}$x$\frac{5}{16}$	----
10" WALL	L5x3$\frac{1}{2}$x$\frac{1}{4}$(*) + L4x3$\frac{1}{2}$x$\frac{1}{4}$(*)	L5x3$\frac{1}{2}$x$\frac{1}{4}$(*) + L4x3$\frac{1}{2}$x$\frac{1}{4}$(*)	L5x5x$\frac{5}{16}$(*) + L4x4x$\frac{5}{16}$(*)	----
12" WALL	⅃LL3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{5}{16}$	⅃LL4x3$\frac{1}{2}$x$\frac{5}{16}$	⅃LL5x3$\frac{1}{2}$x$\frac{5}{16}$	----

NOTES:

1. PROVIDE MINIMUM 6" BEARING ON BRICK, SOLID OR GROUTED SOLID CONCRETE BLOCK.
2. THIS SCHEDULE IS FOR THOSE OPENINGS NOT SHOWN ON THE STRUCTURAL DRAWINGS. REFER TO ARCH. & MECH. DRAWINGS FOR LOCATION AND SIZE OF OPENINGS FOR NON–BEARING MASONRY WALLS.
3. ALL EXTERIOR LINTELS SHALL BE HOT DIP GALVANIZED OR COLD GALVANIZED W/ ZRC GALVANIZING COMPOUND.
4. ALL ANGLES LONG LEG VERT. UNLESS NOTED BY (*). WHEN NOTED BY (*) USE LONG LEG HORIZ.
5. SEE LINTEL DETAIL "3".

MISCELLANEOUS MASONRY LINTEL SCHEDULE FOR NON–LOAD BEARING WALLS

LINTEL SCHEDULE					
MARK	SIZE	B PL (txaxb)	MAX. M.O.	REMARKS	MARK
L1	⅃LL3$\frac{1}{2}$x4x$\frac{5}{16}$	----	4'–0"	SEE "TYP. LINTEL DETAIL 1"	L1
L2	⅃L3$\frac{1}{2}$x4x$\frac{5}{16}$	----	4'–0"	SEE "TYP. LINTEL DETAIL 2"	L2
L3	⅃LL3$\frac{1}{2}$x6x$\frac{5}{16}$	----	5'–6"	SEE "TYP. LINTEL DETAIL 1"	L3
L4	⅃L3$\frac{1}{2}$x6x$\frac{5}{16}$	----	5'–6"	SEE "TYP. LINTEL DETAIL 2"	L4

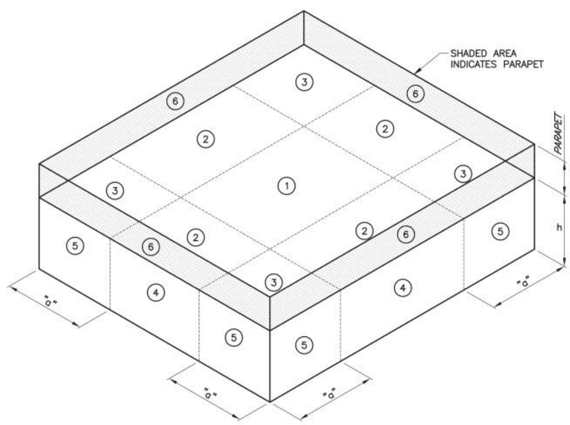
LINTEL NOTES:

1. PROVIDE MINIMUM 6" BEARING ON LOAD BEARING BRICK OR SOLID CONCRETE BLOCK @ EACH END.
2. ALL EXTERIOR LINTELS SHALL BE HOT DIP GALVANIZED.
3. ALL ANGLES LONG LEG VERT. UNLESS NOTED BY (*). WHEN NOTED BY (*) USE LONG LEG HORIZ.
4. FOR LINTEL BEAMS OVER 8" IN DEPTH, PROVIDE MASONRY ANCHORS FROM BEAM WEB TO MASONRY @ 8" O.C. VERT. & @ 16" O.C. HORIZ.
5. SIZE OF LINTEL OPENING AND BEARING ELEVATION TO BE COORD. W/ ARCH. DWGS.

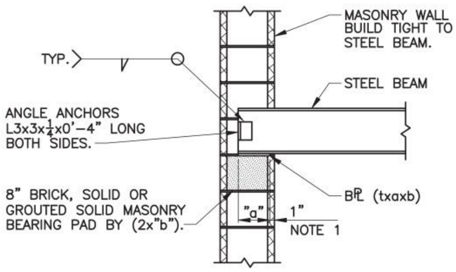
COMPONENT AND CLADDING WIND PRESSURES						
TRIBUTARY AREA (SF)	ROOF ZONE			WALL ZONE		PARAPET
	1	2	3	4	5	6
10	–35	–54	–55	+24/–28	+24/–35	+71/–71
20	–33	–53	–52	+22/–27	+22/–32	+67/–67
50	–30	–48	–48	+21/–25	+21/–29	+62/–62
100	–28	–46	–45	+20/–24	+20/–27	+58/–58
200	–26	–43	–43	+20/–23	+20/–25	+54/–54
500	–24	–39	–39	+17/–21	+17/–21	+49/–49

NOTES:

1. ALL LOADS ARE IN POUNDS PER SQUARE FOOT (PSF).
2. (+) DENOTES PRESSURE, (–) DENOTES SUCTIONS.
3. "a" SHALL BE 10% OF LEAST HORIZ. DIMENSION OR 0.4h, WHICHEVER IS SMALLER, BUT NOT LESS THAN 4% OF LEAST HORIZ. DIMENSION OR 3'–0".



ROOF AND WALL ZONES

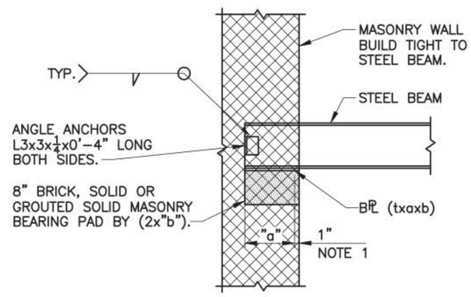


NOTES:

1. FOR BPL'S THAT ARE 1" SMALLER THAN THE MASONRY WALL, CENTER THE BPL ON THE WALL.

TYPICAL STEEL BEAM BEARING ON MASONRY WALL DETAIL

ALTERNATE DETAIL:
PROVIDE 2–$\frac{1}{2}$"ø ANCHOR BOLTS INTO GROUTED SOLID MASONRY BEARING W/ NO ANGLE ANCHORS.



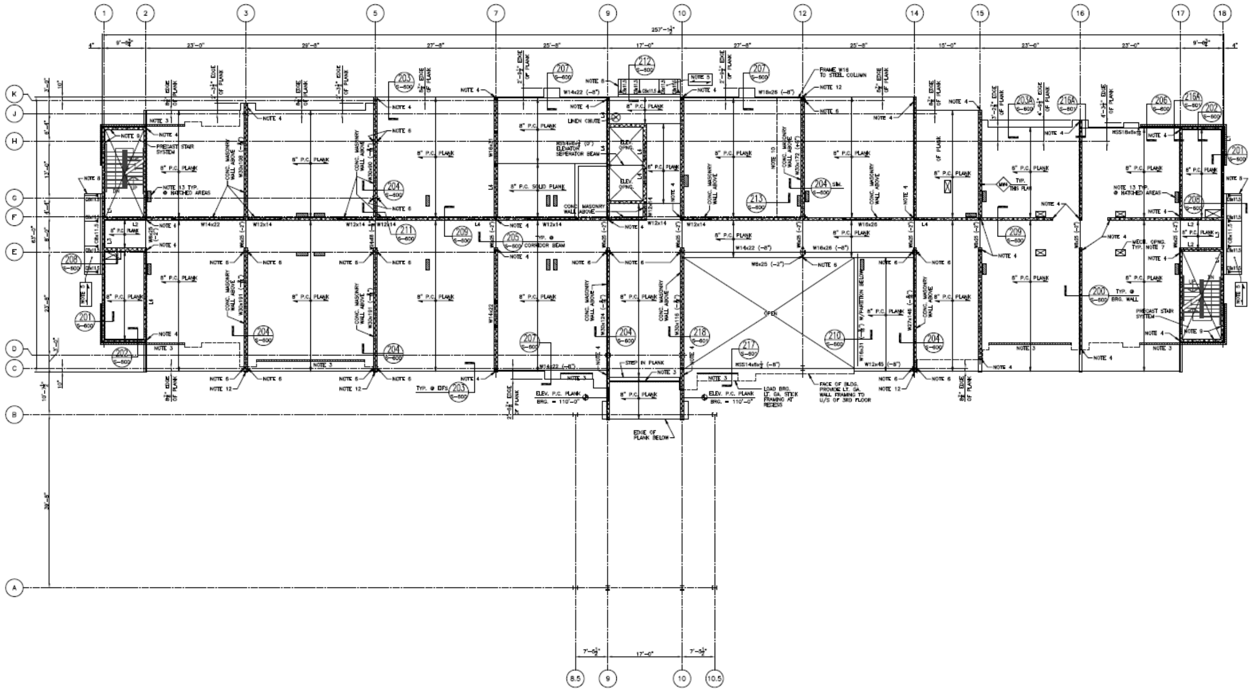
NOTES:

1. FOR BPL'S THAT ARE 1" SMALLER THAN THE MASONRY WALL, CENTER THE BPL ON THE WALL.

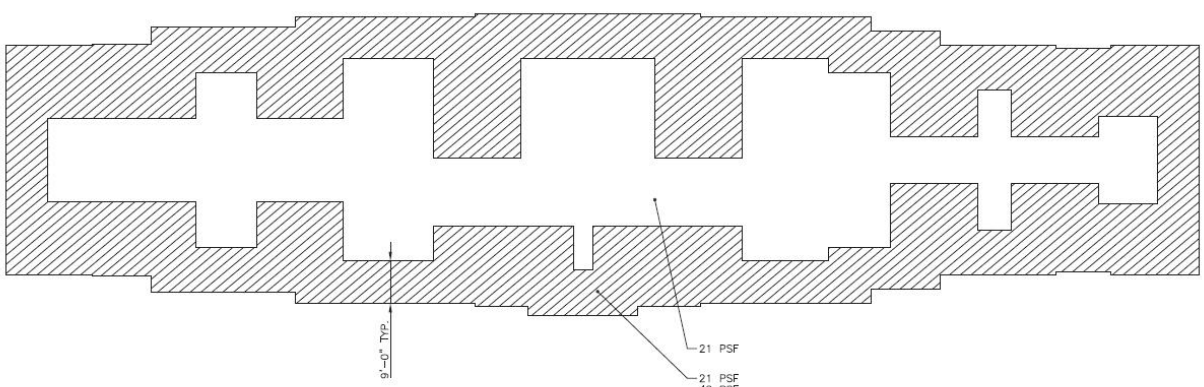
TYPICAL STEEL BEAM BEARING ON MASONRY END WALL DETAIL

ALTERNATE DETAIL:

PROVIDE 2–$\frac{1}{2}$"ø ANCHOR BOLTS INTO GROUTED SOLID MASONRY BEARING W/ NO ANGLE ANCHOR.

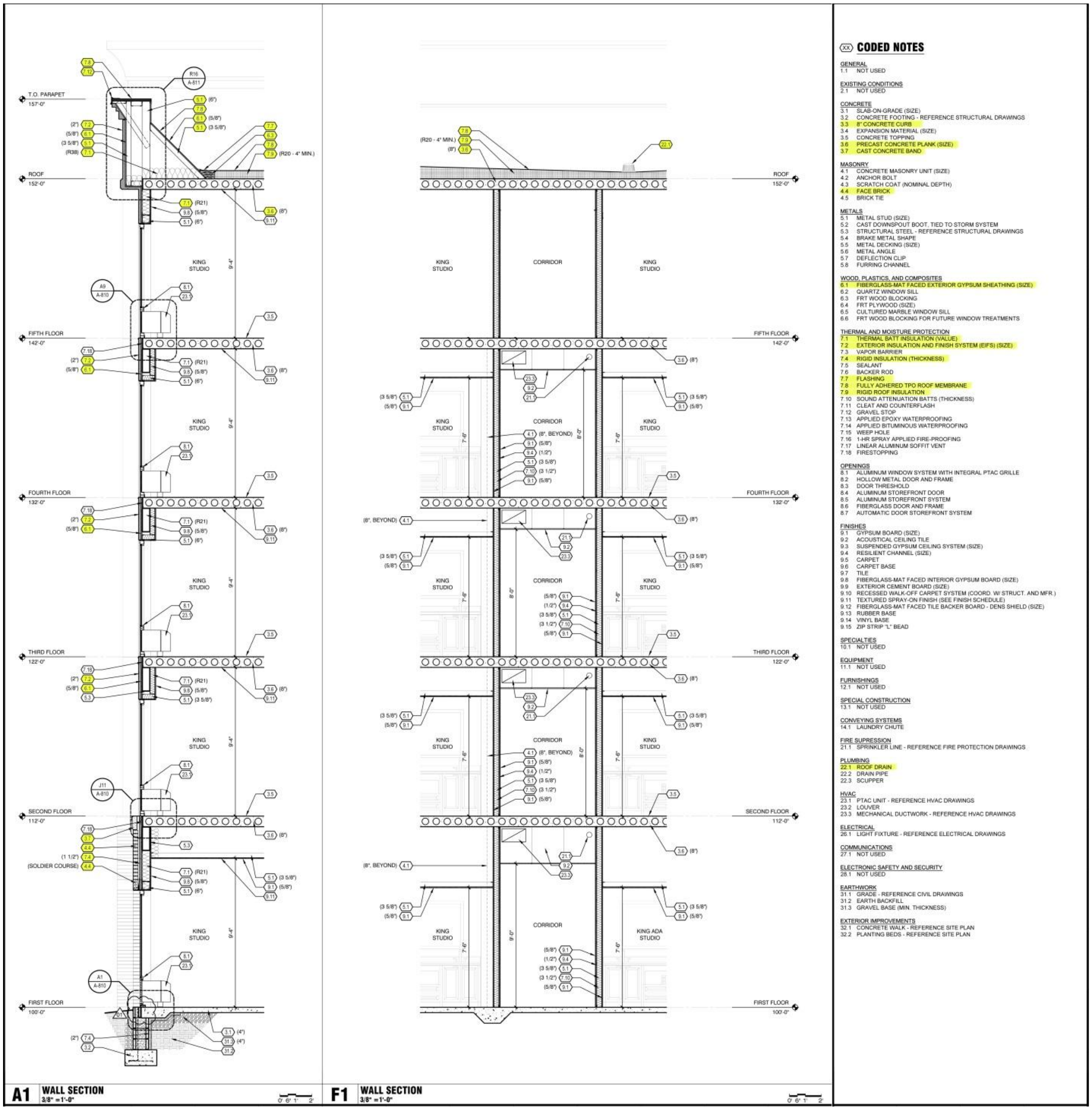


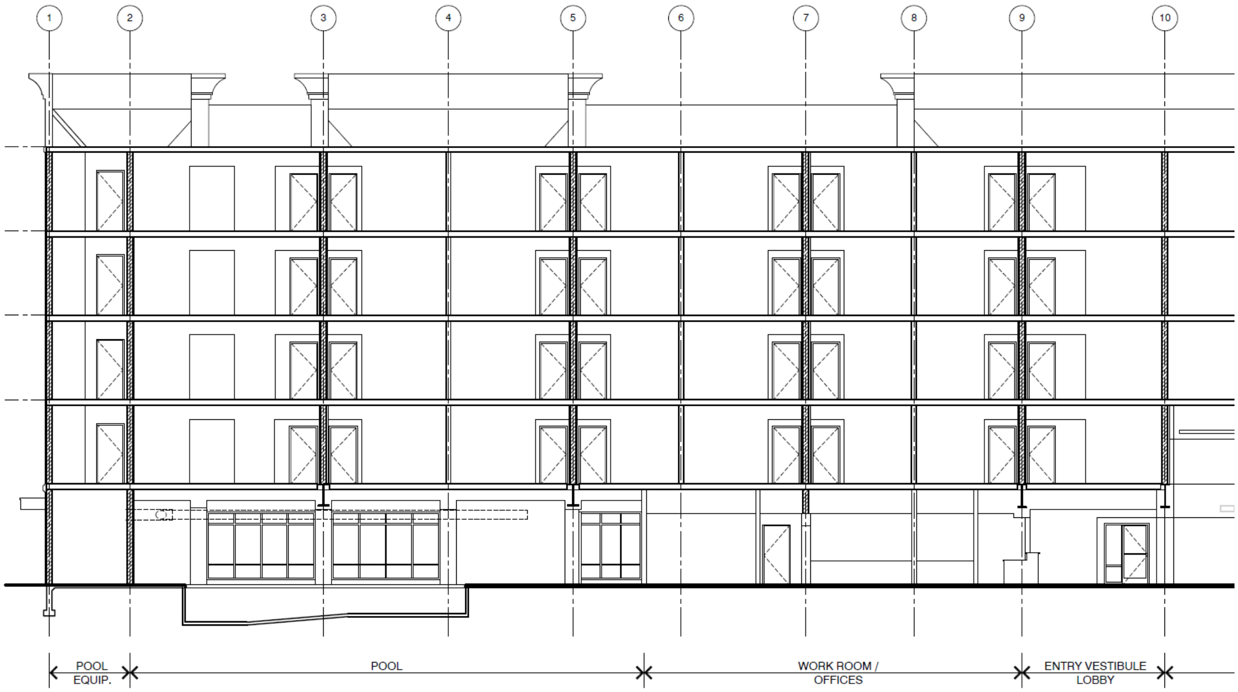
SECOND FLOOR FRAMING PLAN
SCALE: 1/8" = 1'-0"



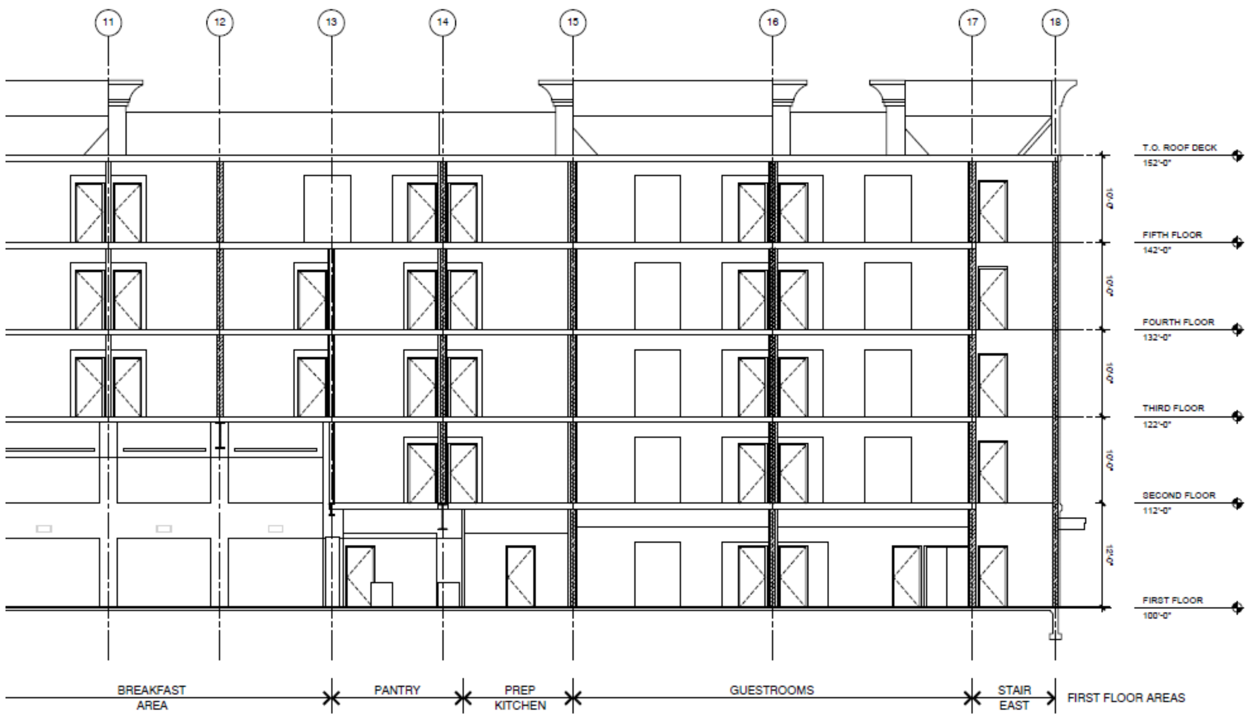
ROOF SNOW LOAD DIAGRAM
SCALE: 1/16" = 1'-0"

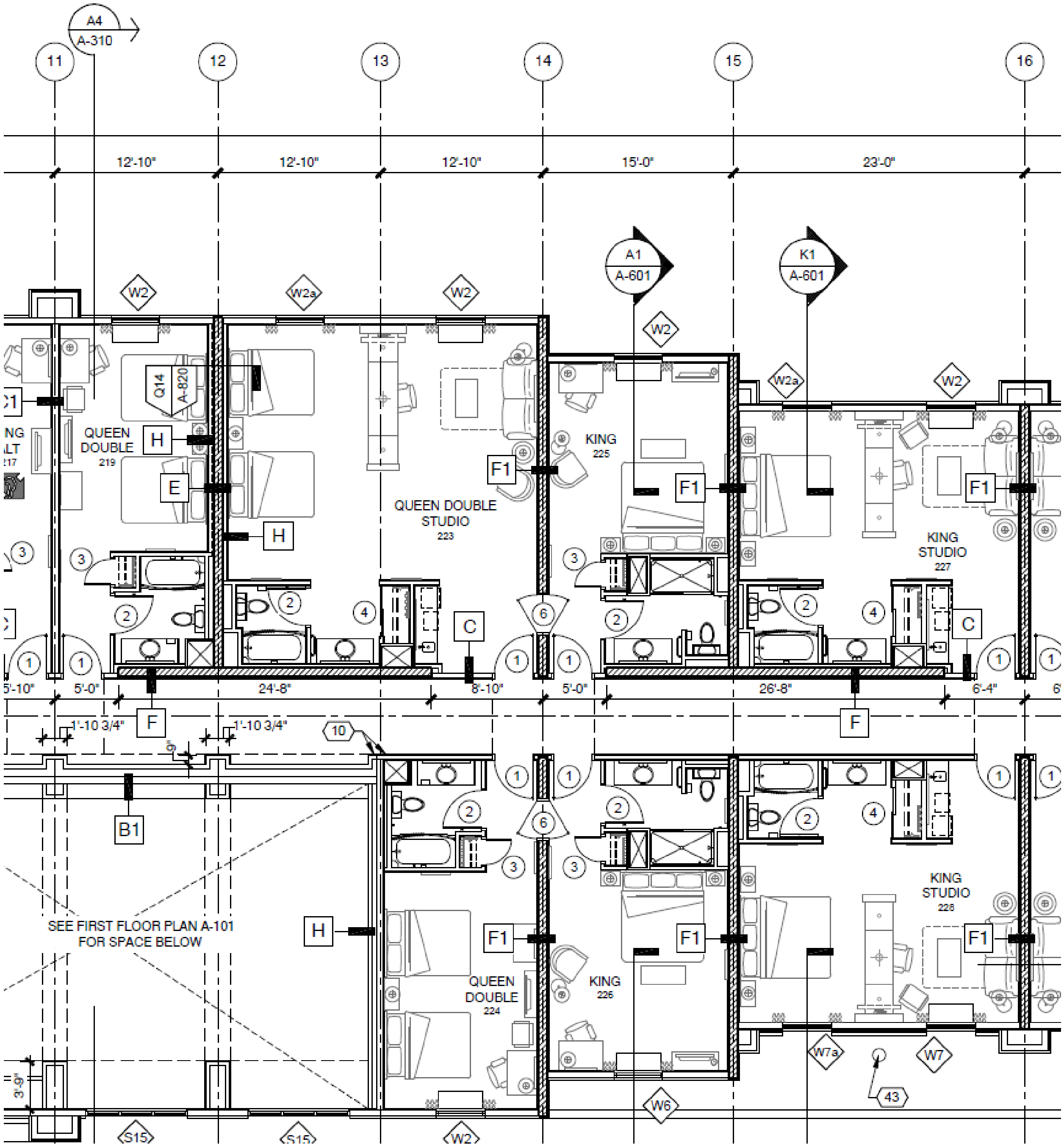
- ✓ IBC 2009
- ✓ International Mechanical Code (IMC 2009)
- ✓ International Plumbing Code (IPC 2009)
- ✓ International Fire Code (IFC 2009)
- ✓ National Fire Protection Associations (NFPA)
- ✓ ADA Accessibility Guidelines (ADAAG) and American National Standards Institute (ANSI)

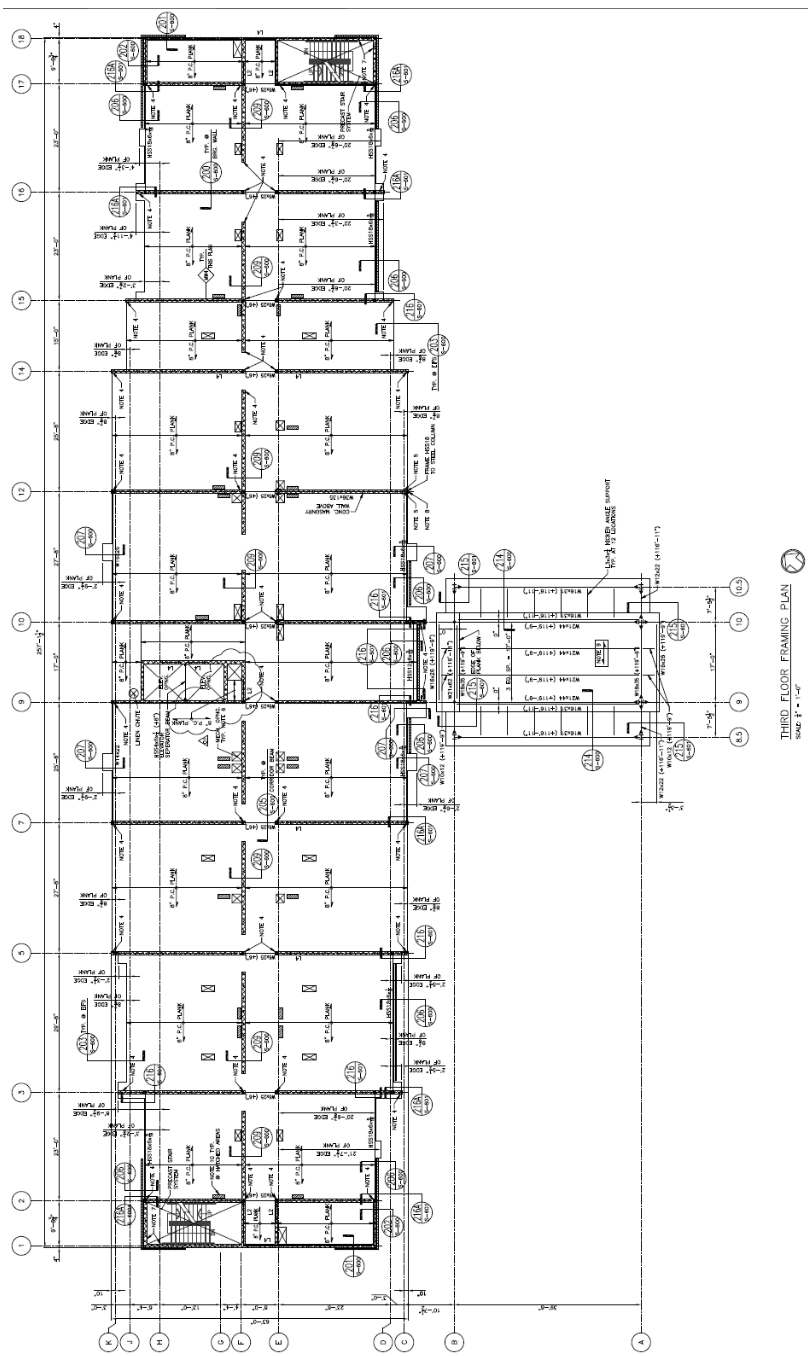


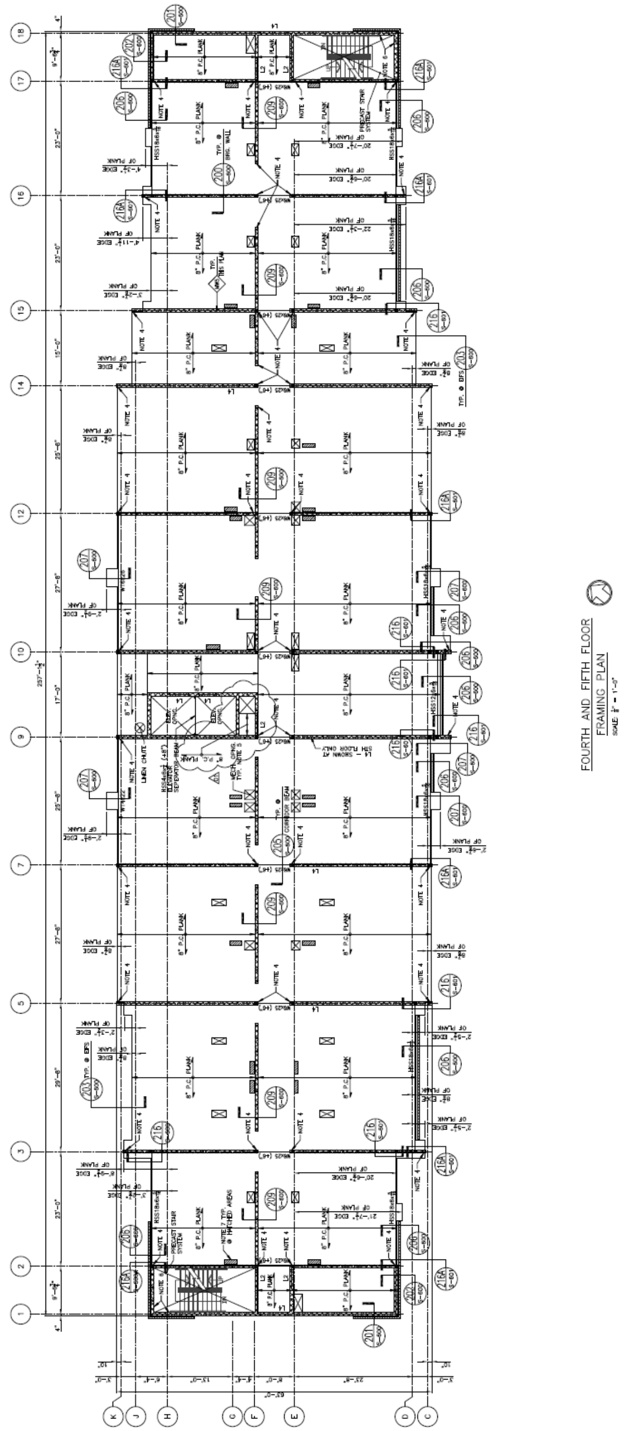


A12 EAST-WEST BUILDING SECTION
1/8"=1'-0"



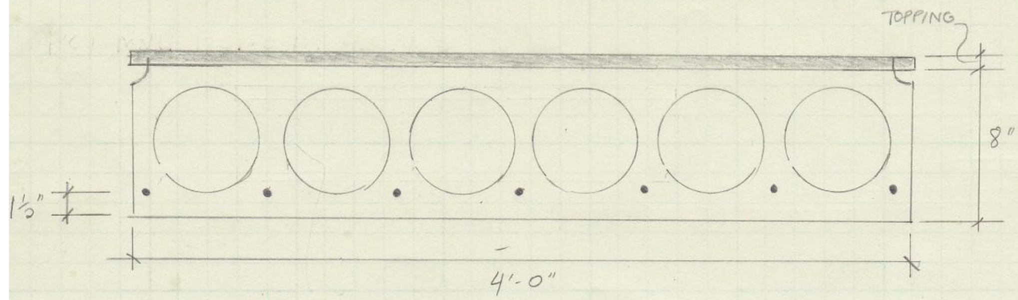

11
12
13
14
15
16
A4
A-310
12'-10"
12'-10"
12'-10"
15'-0"
23'-0"
A1
A-601
K1
A-601
QUEEN
DOUBLE
219
QUEEN DOUBLE
STUDIO
223
KING
225
KING
STUDIO
227
QUEEN
DOUBLE
224
KING
226
KING
STUDIO
228
SEE FIRST FLOOR PLAN A-101
FOR SPACE BELOW
24'-8"
8'-10"
5'-0"
26'-8"
6'-4"
1'-10 3/4"
1'-10 3/4"


THIRD FLOOR FRAMING PLAN


FOURTH AND FIFTH FLOOR FRAMING PLAN

Appendix B: Gravity Load Calculations and Checks

TECH 1	PLANK SPOT CHECK	1	JORDAN RUTHERFORD



$f'_c = 5000\ psi$ $\qquad A = 215\ in^2$

$f'_{pu} = 270000\ psi$ $\qquad I = 1666\ in^4$

$y_b = 4''$

$y_t = 4''$

CHECK 6 STRAND $\qquad S_b = 417\ in^3$

$S_t = 417\ in^3$

$n = 6$ $\qquad wt = 224\ plf$

DIA. = 6/16" $\qquad DL = 56\ psf$

A_{STRAND} $0.085\ in^2$ $\qquad V/S = 1.92\ in$

- LOADS APPLIED: GUEST ROOM 223 (SERVICE)

LIVE LOAD:	40 psf
PARTITIONS:	20 psf
MEP/MISC:	5 psf
CEILING:	3 psf
3/4" TOPPING:	6 psf
SELFWEIGHT:	56 psf
	130 psf

PLANK IS SIMPLE SPAN:

$$M = \frac{w\ell^2}{8} = \frac{520\,(25.66\dot{7})^2}{8} = 42.821\ ft\text{-}k$$

$$M = 42.821\ ft\text{-}k \cdot \frac{12''}{ft} = 513.86\ k\text{-}in$$

$$w = (130\ psf)(4') = 520\ plf$$

- EFFECTS WITHOUT PRESTRESSING

$$f_t = \frac{M}{S_t} = \frac{513.83\ k\text{-}in}{417\ in^3} = -1.232\ ksi$$

$$f_b = \frac{M}{S_b} = \frac{513.83\ k\text{-}in}{417\ in^3} = 1.232\ ksi$$

TECH 1	PLANK SPOT CHECK 2	JORDAN RUTHERFORD

- PRESTRESSING:

$P_e = 0.6(270000\,psi)(0.085\,in^2)(6) = 82.62\,k$

EFFECT OF P_e

$\frac{P_e}{A} = \frac{82.62\,k}{215\,in^2} = -0.3843\,ksi$

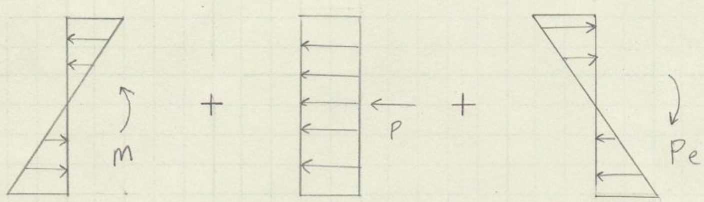
- EFFECTS OF M

$e = 4" - 1.25" = 2.75"$

$P_e(e) = 82.62\,k(2.75") = 227.205\,k\text{-}in$

$f_t = \frac{M}{S} = \frac{227.205\,k\text{-}in}{215\,in^3} = 1.057\,ksi$

$f_b = \frac{M}{S} = \frac{227.205\,k\text{-}in}{215\,in^3} = -1.057\,ksi$



$f_t = -1.232\,ksi - 0.3843\,ksi + 1.057\,ksi = -0.5593\,ksi$

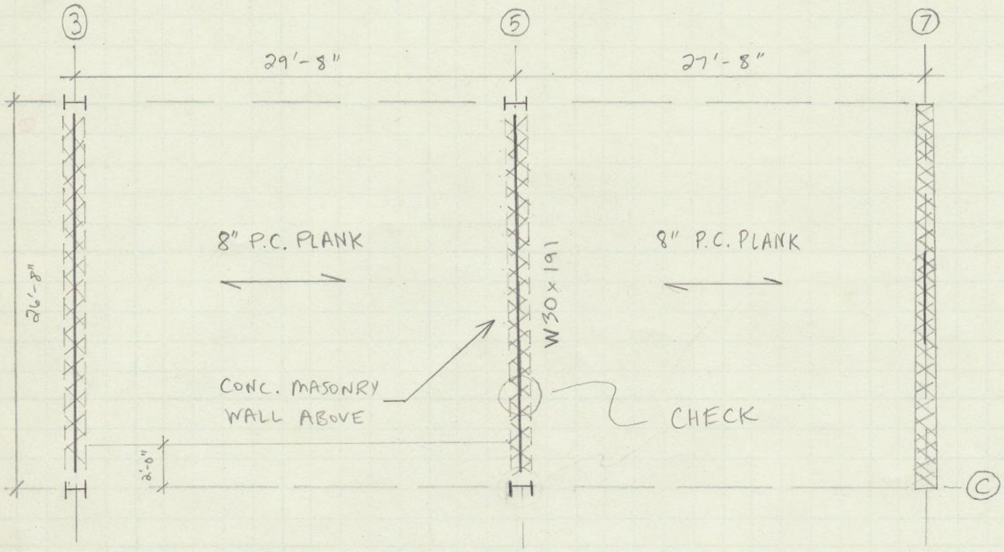
$f_b = 1.232\,ksi - 0.3843\,ksi - 1.057\,ksi = -0.2093\,ksi$

$f_t = -0.5593\,ksi < 3\,ksi = 0.6f'_c$ (ACI 318-11 18.4)

$f_b = -0.2093\,ksi < 0.424\,ksi = 6\sqrt{f'_c}$ (ACI 318-11 18.4)

RESULT: OVERPRESTRESSED, BUT DUE TO ASSUMPTION OF TOPPING ADDING TO LOAD BUT NOT TO SECTION MODULUS, PLANK IS OK ✓

TECH 1	BEAM SPOT CHECK 1	JORDAN RUTHERFORD



BEAM C5-E5

LOADS:

DEAD:

8" PLANK	:	56	psf
3/4" TOPPING	:	6	psf
MEP/MISC.	:	15	psf
8" PLANK	:	56	psf
MEP/MISC	:	10	psf
INSULATION	:	10	psf

LIVE:

GUEST ROOM	:	40 psf
PARTITIONS	:	20 psf
ROOF	:	57 - 21 psf
		USE 36 psf

WALL: 49 psf

TRIBUTARY AREA: $\left(\frac{29.667'}{2}\right)(24.667') + \left(\frac{27.667'}{2}\right)(26.667') = \boxed{734.8\ ft^2}$

LL REDUCTION: $L_{max}\ \begin{cases} L_o\left(.25 + \frac{15}{\sqrt{K_{LL}A_t}}\right) = 60\left(.25 + \frac{15}{\sqrt{1469.6}}\right) = \boxed{38.47\ psf} \\ 0.4L_o \qquad 0.4(60) = 24\ psf \end{cases}$

LINEAR LOAD:

$$w_u = 1.2\left[(3)(77)(42.5) + (76)(42.5) + (49)(40)\right] + 1.6\left[(3)(38.47)(42.5) + (34)(42.5)\right]$$

$$w_u = 18009 + 10295 = 28304\ plf$$

MOMENT:

AISC 14th (TABLE 3-2)

$$M_u = \frac{(25974\ plf)(26.667')^2}{8} = 2316\ ft\text{-}k < 2530\ ft\text{-}k \quad OK$$

TECH 1	BEAM SPOT CHECK 2	JORDAN RUTHERFORD

DEAD/LIVE LOAD DEFLECTION:

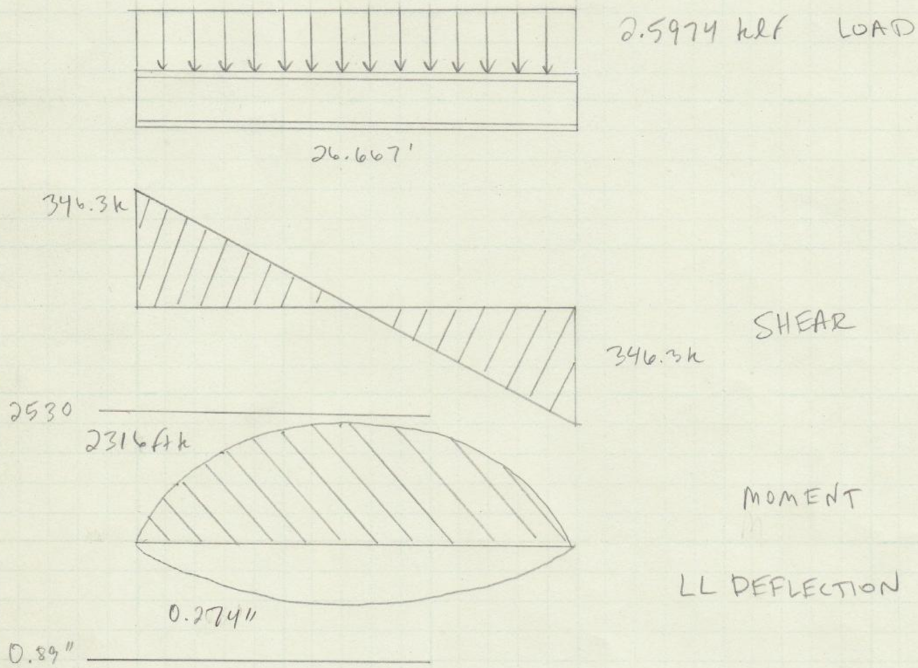
$w_{LL} = (3)(38.47)(42.5) + (36)(42.5) = 6432 \text{ plf}$

$\Delta_{LL} = \dfrac{5 w_{LL} l^4}{384 EI}$

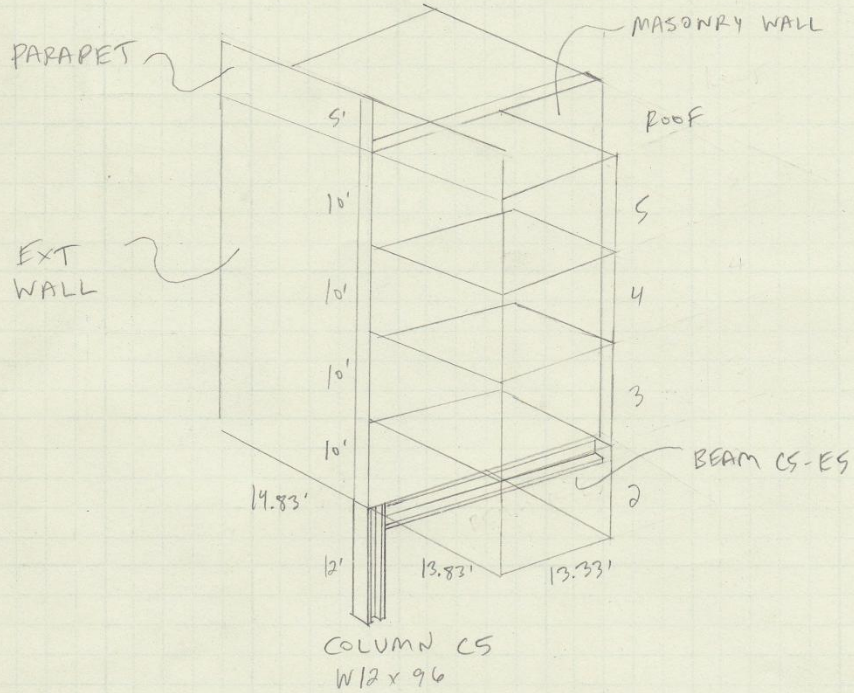
$\Delta_{LL} = \dfrac{5(6.43 \text{ klf})(26.667)^4(1728)}{384(29000 \text{ ksi})(9200 \text{ in}^4)} = 0.274'' < 0.89'' = \dfrac{(26.667')(12)}{360}$

$w_{DL} = (3)(77)(42.5) + (76)(42.5) + (49)(40) = 15008 \text{ plf}$

$\Delta_{DL} = \dfrac{5(15.01)(26.667)^4(1728)}{384(29000)(9200)} = 0.64'' < 1.333'' \quad \dfrac{(26.667')(12)}{240}$



TECH 1	COLUMN SPOT CHECK 1	JORDAN RUTHERFORD



COLUMN C5:

FLOOR LOADS:

DEAD: 8" PLANK : 56 psf
3/4" TOPPING : 6 psf
MEP/MISC : 15 psf
WALL : 49 psf

LIVE: GUEST ROOM : 40 psf
PARTITIONS : 20 psf

ROOF LOADS:

DEAD: 8' P.C. PLANK : 56 psf
8" INSULATION : 10 psf
MEP/MISC : 10 psf

LIVE: 57 — 21
USE 41 psf AVERAGE

EXT WALL: INSULATION : 5 psf
C.F. STUDS : 1.5 psf
16" o.c.

TRIB AREA: $\left(\frac{29.667'}{2}\right)\left(\frac{26.667'}{2}\right)+\left(\frac{27.667'}{2}\right)\left(\frac{26.667'}{2}\right) = 383\ ft^2$

LL REDUCTION: L_{MAX} $\left|\begin{array}{l} 60\left(.25+\frac{15}{\sqrt{765}}\right) = \boxed{47.55\ psf} \\ 0.4(60) = 24\ psf \end{array}\right.$

TECH 1	COLUMN SPOT CHECK 2	JORDAN RUTHERFORD

AXIAL LOAD:

ROOF: $1.2[(76)(383)] + 1.6[(36)(383)] = 56990.4\ k$

FLOORS 2-4: $1.2\left[3(77)(383) + (49)(40)\left(\frac{26.667}{2}\right) + (6.5)(45)\left(\frac{29.667}{2} + \frac{27.667}{2}\right)\right.$
$+$
$1.6[3(47.55)(383)] = 235006\ k$

$P_u = \boxed{292\ k}$ (VERY CLOSE TO 295 k TO BASE PLATE ON COLUMN SCHEDULE)

CHECK BUCKLING:

W12×96 ASSUMING $k=1$

AREA $= 28.2\ in^2$
$r_y = 3.09$

$\frac{kL}{4} = \frac{(1)(12)(12)}{4} = 36 < 113 = 4.71\sqrt{\frac{29000}{50}}$

$$F_e = \frac{\pi^2 E}{\left(\frac{kL}{r_y}\right)^2} = \frac{\pi^2(29000)}{\left(\frac{(1)(12)(12)}{3.09}\right)^2} = 131.8$$

$$F_{cr} = \left[0.658^{50/131.8}\right]50 = 42.65\ ksi$$

$$P_n = A_g F_{cr} = (28.2\ in^2)(42.65\ ksi) = 1202\ k$$

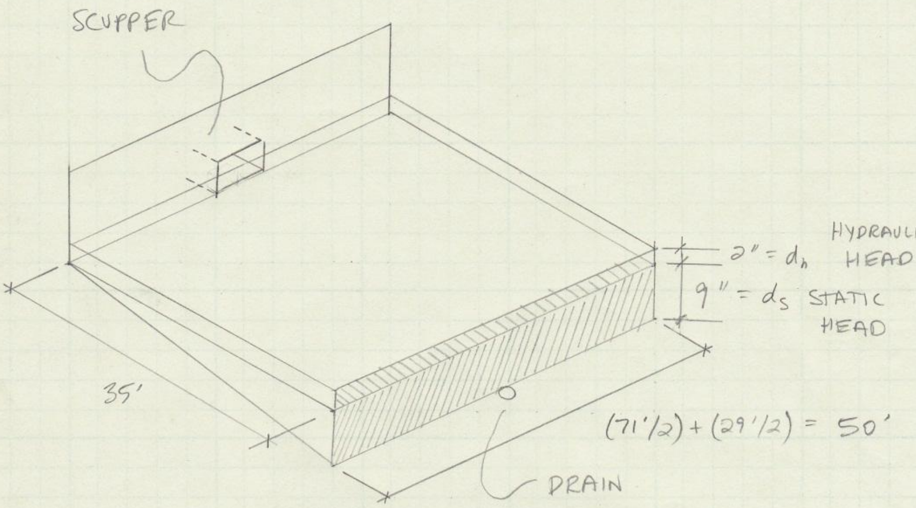
$$\phi P_n = 0.9\,(1202\ k) = \boxed{1082\ k > 288.9\ k}\ \text{OK} \checkmark$$

1080 k
(4-1)

Appendix C: Rain and Snow Load Calculations

TECH 1	RAIN LOAD 1	JORDAN RUTHERFORD

RAIN LOAD PER IBC 2009



SCUPPER SIZE: 8×6 (USE 6×6)

AREA: 35' × 50' = 1750 ft^2

RAINFALL RATE: i = 2.75 "/hr
0.229167 ft/hr

FLOW RATE: $Q = A \times i$

$(1750\ ft^2)(0.229167\ ft/hr)$ = 401 ft^3/hr
6.68 ft^3/min
50 g/min

HYDRAULIC HEAD: d_h = 2" (SEE EXCEL)

RAIN LOAD: 5.2(9" + 2") = $\boxed{57.2\ psf}$

TECH 1 | SNOW LOAD | 1 | JORDAN RUTHERFORD

FLAT ROOF SNOW LOAD:

GROUND SNOW LOAD: (FIG 7-1) 30 psf
IMPORTANCE FACTOR: (TBL 7-4) 1.0
EXPOSURE FACTOR: (TBL 7-2) 1.0 (CONSERVATIVE)
THERMAL FACTOR: (TBL 7-3) 1.0

$p_f = 0.7(1.0)(1.0)(1.0)(30) = \boxed{21\ psf}$

SNOW DRIFT: (7.8 / 7.7.1)

SPECIFIC GRAVITY: $\gamma = 0.13(30) + 14 = 17.9\ pcf < 30\ pcf$ OK ✓

BASE ACCUMULATION: $h_b = p_f / \gamma = 21/17.9 = 1.17'$

$h_c = 8.67' - 1.17' = 7.5'$

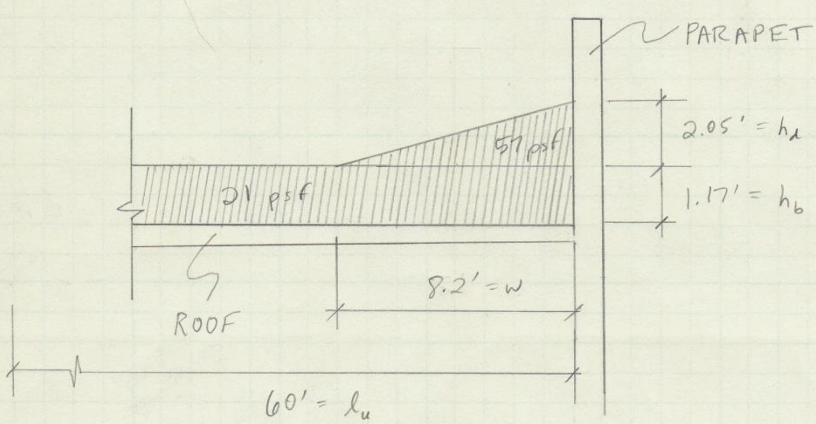
$h_c / h_b = 7.5' / 1.17 = 6.41 > 0.2$ MUST CALC. DRIFT

WINDWARD DRIFT:

$h_d = 0.75(0.43 \sqrt[3]{60} \sqrt[4]{30+10} - 1.5) = 2.05'$

$W = 4(2.05') = 8.2'$

$p_d = (17.9\ pcf)(2.05') = \boxed{57\ psf}$



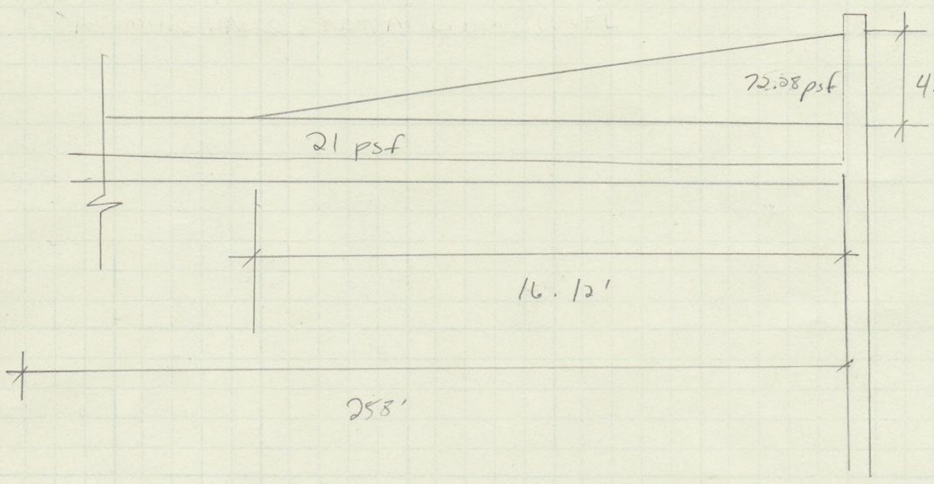
TECH 1 | SNOW LOAD 2 | JORDAN RUTHERFORD

DRIFT ALONG LENGTH

$h_d = 0.75(.43\sqrt[3]{258}\sqrt[4]{30+10} - 1.5) = 4.03'$

$w = 4(4.03') = 16.12'$

$p_d = \gamma(4.03') = 72.28$ psf



Appendix D: Lateral Load Calculations

Masonry Shear Wall Data (C_w) for E-W												
Column Line	t_i (in)	D_i (ft)	A_i (ft)	h_i (ft)	h_n (ft)	Floor	Σ	A_b	100/A_b	Σ	C_w	T_a
1	8.00	40.00	26.67	52.00	52.00	1.00	11.10	15725	0.006359	103.65	0.659153	**0.1217**
2	8.00	40.00	26.67	52.00	52.00	1.00	11.10					
7	8.00	41.27	27.51	52.00	52.00	1.00	11.87					
9	8.00	30.96	20.64	52.00	52.00	1.00	6.17					
10	8.00	38.79	25.86	52.00	52.00	1.00	10.38					
14	8.00	26.67	17.78	52.00	52.00	1.00	4.28					
15	8.00	47.55	31.70	52.00	52.00	1.00	15.91					
16	8.00	39.75	26.50	52.00	52.00	1.00	10.95					
17	8.00	39.75	26.50	52.00	52.00	1.00	10.95					
18	8.00	39.75	26.50	52.00	52.00	1.00	10.95					
						Σ	103.65					
Masonry Shear Wall Data (C_w) for N-S												
Column Line	t_i (in)	D_i (ft)	A_i (ft)	h_i (ft)	h_n (ft)	Floor	Σ	A_b	100/A_b	Σ	C_w	T_a
F	8.00	70.50	47.00	52.00	52.00	1.00	32.38	15725	0.006359	239.6828	1.524215	**0.08**
						Σ	239.6828					

Masonry Wall Weight (tek 14-3b)							
Type	Width	Vertical Reinforcing	Weight (psf)	Length (ft)	Height (ft)	Floor	Weight (k)
Masonry Wall 1	8"	#5 @ 24" O.C.	47	525	6	G	148.05
			47	798	10	2	1500.24
183			47	721	10	3	1355.48
			47	721	10	4	1355.48
			47	721	10	5	1355.48
Masonry Wall 2	8"	#5 @ 24" O.C.	47	161	6	G	45.40
			47	161	10	2	75.67
			47	161	10	3	75.67
			47	161	10	4	75.67
			47	161	10	5	75.67
Masonry Wall 3	12"	#5 @ 48" O.C.	53	499	6	G	158.68
Masonry Wall 4	8"	#5 @ 24" O.C.	47	26	10	3	12.22
Masonry Wall 5	8"	#5 @ 32" O.C.	43	26	10	4	11.18
			43	26	10	5	11.18

Total	G	352.13
	2	1575.91
	3	1443.37
	4	1442.33
	5	1442.33

Floor Dead Loads	Load (psf)	Reference
8" Precast Plank	56	PCI MNL 120
3/4" Topping	6	DATA FROM AES
Paritions	10	12.14.8.1
MEP/Misc.	5	
Ceiling	3	
Total	80	
Roof Dead Load	**Load (psf)**	**Reference**
8" Precast Plank	56	PCI MNL 120
MEP/Misc.	5	
Ceiling	3	
Insulation	12	DATA FROM AES
Total	76	

Total Building Weight				
Level	Area (ft^2)	Load (k)	Wall Weight (k)	Total (k)
Ground	15725	0	352.13	352.13
2	13133	1051	1575.91	2626.55
3	14370	1150	1443.37	2592.97
4	14370	1150	1442.33	2591.93
5	14370	1092	1442.33	2534.45
			Total Weight(k)	**10698.03**

TECH 1 | SEISMIC ANALYSIS 1 | JORDAN RUTHERFORD

EQUIVALENT LATERAL FORCE METHOD

OCCUPANCY CATEGORY: (TBL 1-1) II

SITE CLASS: (GEOTECH. REPORT) D

SEISMIC LOAD IMPORTANCE FACTOR: (FIG 11.5-1) $I_e = 1.0$

SPECTRAL RESPONSE ACCELERATIONS: (FIG 22-1,2) $S_S = 0.125$
$S_1 = 0.049$

SITE CLASS COEFFICIENT: (TBL 11.4-1,2)

$F_a = 1.6 \quad S_{MS} = 1.6\,(0.125) = 0.2$
$F_v = 2.4 \quad S_{M1} = 2.4\,(0.049) = 0.1176$

SPECTRAL RESPONSE COEFFICIENT: (TBL 11.4-3,4)

$S_{DS} = 2/3\,(0.2) = 0.1333$
$S_{D1} = 2/3\,(0.1176) = 0.0784$

SEISMIC DESIGN CATEGORY: (TBL 11.6-1,2) B

BASE SEISMIC FORCE RESISTING SYSTEM: (TBL 12.2-1) $R = 2$
REINFORCED MASONRY SHEAR WALLS

APPROXIMATE FUNDAMENTAL PERIOD: (12.8.2.1)

$T_a = C_t h_n^x = 0.02\,(52)^{0.75} = 0.387$ FOR "OTHER" SYSTEMS (TBL 12.8-1)

OR

$T_a = \dfrac{0.0019}{\sqrt{C_W}}\,h_n = $ E-W: 0.1217 | N-S: 0.08
FOR MASONRY SHEAR WALLS 12.8-9

$$C_W = \frac{100}{A_B}\sum_{i=1}^{x}\left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]} = 0.659 \;/\; 1.524$$

$A_b = 15725\ ft^2$

SEE EXCEL FOR A_i, D_i, h_i, t_i

TECH 1 | SEISMIC ANALYSIS 2 | JORDAN RUTHERFORD

SEISMIC RESPONSE COEFFICIENT: (12.8.1.1)

COEF. FOR UPPER LIMIT ON PERIOD: (12.8-1)

$C_u = 1.7 \times T_a = 0.387 = T = 0.6579$

$T_L = 12\,s$ (FIG 22-15)

$$C_s = \frac{S_{DS}}{\frac{R}{I}} = \frac{0.1333}{\left(\frac{2}{1}\right)} = 0.067$$

$$C_{s\,max} \begin{cases} \dfrac{S_{D1}}{T\left(\frac{R}{I}\right)} & \dfrac{0.0784}{0.6579\left(\frac{2}{1}\right)} = 0.059 \quad \text{FOR } T < T_L \text{ OK} \\[2ex] \dfrac{S_{D1} T_L}{T^2\left(\frac{R}{I}\right)} & \dfrac{0.0784\,(12)}{(0.6579^2)\left(\frac{2}{1}\right)} = 1.087 \quad \text{FOR } T > T_L \text{ NG} \end{cases}$$

$$C_{s\,min} \begin{cases} \dfrac{0.5\,S_1}{\left(\frac{R}{I}\right)} = \dfrac{0.5\,(0.049)}{\left(\frac{2}{1}\right)} = 0.01225 \\[2ex] 0.01 \end{cases}$$

SEE EXCEL TABLE FOR DETAILED C_s PER DIR.

TECH 1	SEISMIC ANALYSIS 3	JORDAN RUTHERFORD

BASE SHEAR: (12.8.1)

<u>DEAD LOAD:</u>

P.C. PLANK: 56 psf
3/4" TOPPING: 6 psf
PARTITIONS: 15 psf (4.2.2)
MEP/MISC: 5 psf
CEILING: 3 psf
85 psf

<u>ROOF:</u>

P.C. PLANK: 56 psf
MEP/MISC: 5 psf
CEILING: 3 psf
INSULATION: 12 psf
76 psf

FLOOR AREAS	WEIGHT	WALL WEIGHT
2: 14871 ft²	1264 k	COMPLETED
3: 14871 ft²	1264 k	IN
4: 14871 ft²	1264 k	EXCEL
5: 14871 ft²	1130 k	

$$V = C_s W = 0.067\,(10907) = 649$$

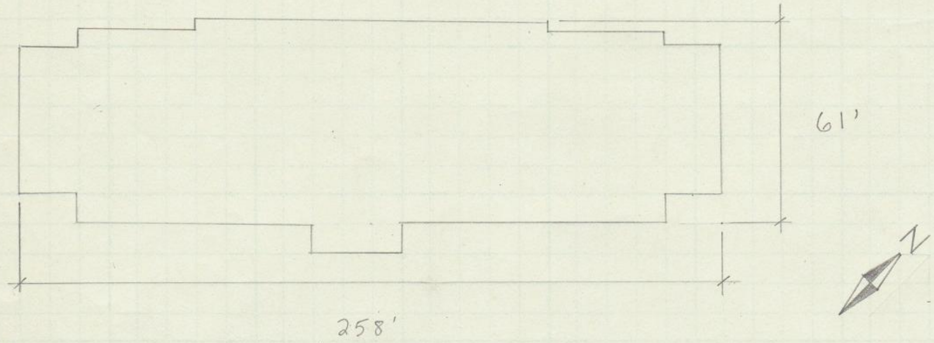
VERTICAL FORCE DISTRIBUTION: (12.8.3)

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^{\,k}}{\sum_{i=1}^{n} w_i h_i^{\,k}}$$

CALCULATIONS DONE IN EXCEL

TECH 1 | WIND ANALYSIS 1 | JORDAN RUTHERFORD



PARAPET HEIGHT: 60'8" USE METHOD 2
E-W DIRECTION: 258'
N-S DIRECTION: 61'

WIND SPEED: (FIG 6-1) $V = 90$ MPH

DIRECTIONALITY FACTOR: (TBL 6-4) $K_d = 0.85$

OCCUPANCY CATEGORY: (TBL 6-1) II

IMPORTANCE FACTOR: (TBL 6-1) $I = 1.0$

EXPOSURE CATEGORY: (6.5.6.3) C

TOPOGRAPHIC FACTOR: (FIG 6-4) $K_{zt} = 1.0$

VELOCITY PRESSURE COEFFICIENTS: (TBL 6-3) VARIES W/ HEIGHT

INTERNAL PRESSURE COEFFICIENT: $GC_{pi} = \pm 0.18$

GUST FACTOR: (6.5.8.1) $G_f = 0.85$*

* CALCS PERFORMED BASED ON RIGID STRUCTURE, 0.85 USED TO BE CONSERVATIVE

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{33}{0.6(60.67)}\right)^{1/6} = .197$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{0.6(60.67)}{33}\right)^{1/5} = 509.9$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{258+60.67}{509.9}\right)^{.63}}} = .825 \qquad G_f = 0.925\left(\frac{1+(1.7)(3.4)(.197)(.825)}{1+(1.7)(3.4)(.197)}\right) = .8386$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{61+60.67}{509.9}\right)^{.63}}} = .892 \qquad G_f = 0.925\left(\frac{1+(1.7)(3.4)(.197)(.892)}{1+(1.7)(3.4)(.197)}\right) = .872$$

TECH 1	WIND ANALYSIS 2	JORDAN RUTHERFORD

BUILDING IS FULLY ENCLOSED

WALL PRESSURE COEFFICIENTS: (FIG 6-6)

	C_p		USE WITH
WINDWARD:	0.8		q_z
SIDEWALL:	-0.5	(L/B < 1) (FOR N-S)	q_h
	-0.2	(L/B > 4) (FOR E-W)	q_h
LEEWARD:	-0.7		q_h

ROOF PRESSURE COEFFICIENTS (FIG 6-6) q_h

		C_p	
WINDWARD:	0 - h/2	-0.9	2h = 121' < 258'
	h/2 - h	-0.9	∴ USE ALL
$h/L = \frac{60.67}{258} = .22 < .5$	h - 2h	-0.5	h/2 = 30.33'
	> 2h	-0.3	

			AREA
WINDWARD:	0 - h/2	-1.3	(30.33)(258') = 7825 > 1000 X
	> h/2	-0.7 ← REDUCE X 0.8	(30.67)(258') = 7912 > 1000 ✓
$h/L = \frac{60.67}{61} = .99 > .5$			

VELOCITY PRESSURES: $q_z = 0.00256 K_z K_{zt} K_d V^2 I$

$q_h = 0.00256 K_z K_{zt} K_d V^2 I$

DESIGN WIND PRESSURES: (6.5.12.2)

WINDWARD: $p_z = q_z(GC_p) - q_h(GC_{pi})$

LEEWARD:
SIDEWAYS
ROOF
$p_h = q_h(GC_p) - q_h(GC_{pi})$

PARAPET: $p_p = q_p(GC_{pn}) = q_p(1.5)$ WINDWARD
$q_p(-1.0)$ LEEWARD

☆ FOR VALUES, SEE EXCEL TABLES